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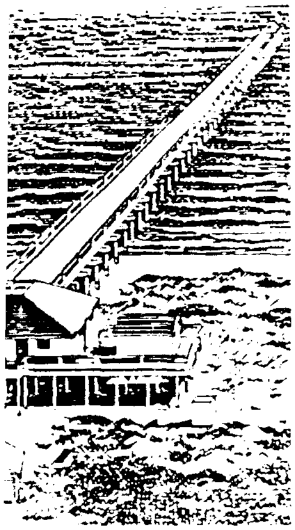

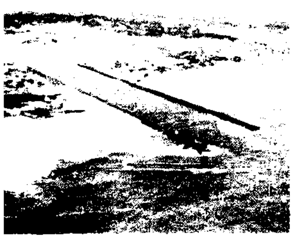

MONITORING COMPLETED COASTAL PROJECTS PROGRAM

MISCELLANEOUS PAPER CERC-91-8

MONITORING OF JETTY REHABILITATION AT MANASQUAN INLET, NEW JERSEY

by

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13. ABSTRACT *(Maximum 200 words)*

Under the Monitoring Completed Coastal Projects Program, an assessment of jetty rehabilitation at Manasquan Inlet, New Jersey, was performed. Objectives of the monitoring effort were to evaluate the performance of the dolos armor units in maintaining structural stability of the jetties, determine potential effects of the rehabilitated jetties on longshore sediment movement at the inlet, and determine the effectiveness of the rehabilitated jetties in maintaining a stable inlet cross section. Data collection for the monitoring program occurred from June 1982 to October 1984. Data on the performance of the project, observations noted in the prototype, and results of the data collection, as well as conclusions and recommendations based on the monitoring effort, are reported herein.

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currents	Manasquan Inlet, New Jersey	structural stability
dolosse	photogrammetry	tides
jetties	side scan sonar	waves

Contents

Preface

Funding for the study reported herein was provided through the Monitoring Completed Coastal Projects (MCCP) Program. The program entails intense monitoring of selected Civil Works coastal projects to assure adequate information as a basis for improving project purpose attainment, design procedures, construction methods, and operation and maintenance techniques. Overall program management is by the Hydraulic Design Section of Headquarters, US Army Corps of Engineers (HQUSACE). The Coastal Engineering Research Center (CERC), US Army Engineer Waterways Experiment Station (WES), is responsible for technical and data management and support for HQUSACE review and technology transfer. Technical Monitors for the MCCP Program are Messrs. John H. Lockhart, Jr.; John G. Housley; James E. Crews; and Robert H. Campbell. The Program Manager is Ms. Carolyn M. Holmes, CERC.

Acknowledgments are extended to the following: Mr. R. Ray Bottin, Jr., CERC, for all his contributions in organizing and preparing this report; Ms. Lee Byrne, Information Technology Laboratory, WES, for editing this report; and Ms. Karen Wood, CERC, for typing this report.

This report was written by Messrs. Jeffrey Gebert, US Army Engineer District, Philadelphia, and J. Michael Hemsley, former MCCP Program Manager, under the general supervision of Mr. Charles C. Calhoun, Jr., and Dr. James R. Houston, Assistant Chief and Chief of CERC, respectively.

Commander and Director of WES during the report publication was COL Larry B. Fulton, EN. Technical Director was Dr. Robert W. Whalin.

Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
cubic feet	0.02831685	cubic metres
cubic yards	0.7646	cubic metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
inches	2.54	centimetres
miles (US statute)	1.609344	kilometres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square feet	0.09290304	square metres
tons (2,000 pounds, mass)	907.1847	kilograms

1 Introduction

History

Manasquan Inlet is located on the Atlantic Coast of New Jersey approximately 26 miles[1] south of Sandy Hook and 23 miles north of Barnegat Inlet (Figure 1). The inlet provides the northernmost connection between the ocean and the New Jersey Intracoastal Waterway. Reliable surveys as early as 1839 show that the inlet has migrated between its present location and 1 mile north. On a number of occasions prior to jetty completion in 1931, the inlet closed completely.

Stabilization of the inlet was first attempted between 1881 and 1883 with the construction of timber jetties. Both these and subsequent Haupt reaction jetties built in 1922 failed, leading to Congressional authorization of the present project layout in 1930. The project involved construction of two rubble jetties, with steel sheet-pile cores, spaced 400 ft apart. Built to a crest height of +14 ft mean low water (MLW),[2] the jetties extended to the -10 ft contour. Core stone weight ranged from 100 to 500 lb; 2-ton capstone was used for armor. Originally, the authorized channel was 250 ft wide and 10 ft deep between the jetties and 300 by 8 ft for the interior channels. In 1935, the authorized channel depth between the jetties was increased to 14 ft and the interior channel depth to 12 ft.

Through the mid-1970s, the jetties were repeatedly damaged by storms and structural settlement (US Army Engineer District (USAED), Philadelphia 1978). Beach erosion north of the inlet and accretion south emphasized the impact of the jetties on the littoral system. Shoaling of the navigation channel increased as the structures deteriorated and became more permeable. Numerous repairs were attempted, using armor stone of up to 12 tons, without success (Figure 2).

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1 A table of factors for converting non-SI units of measurement to SI units is presented on page vii.

2 All elevation (el) and depths cited herein are in feet referred to MLW, unless otherwise noted.

Design of the Rehabilitation

The most recent rehabilitation of the jetties was completed in 1982 and involved the use of 16-ton dolosse as armor. Work was completed on the 1,000-ft-long south jetty in 1980. Inshore of the landward end of the jetty, the channel is protected by a sheet-pile bulkhead. The first step in the rehabilitation of the jetties was to disassemble them. Sand and dislodged armor stone were excavated and reshaped to the design configuration prior to dolos placement (Figure 3). Dolosse were placed on the outer 400-ft of the north or channel side of the jetty, around the structure head, and along the outer 120 ft of the south side. Dolosse extended to -10 ft on the channel side at a slope of one vertical on two horizontal (1v:2h). Inshore of the dolos section, the side slopes were armored with a single layer of 12-ton stones. The outer 400 ft of the jetty crest is a concrete cap; the inner 600 ft of crest is 12-ton stone. The original sheet-pile core was left in place in its existing condition. The sheet pile extends the entire length of the jetty and has a top elevation of +8 ft.

Rehabilitation of the 1,200-ft-long north jetty began in 1980 and was completed in 1982. Dolosse were placed along the outer 250-ft of the jetty on its north side, around its head, and along the outer 90 ft on the channel side. Stone was used to armor the inner ends of the jetty on both sides near the dolosse with stone size decreasing to 3 to 5 tons at the shore ends. Construction techniques were the same for both jetties. Construction drawings of typical cross sections for the jetties are shown in Figures 4 and 5.

Breaking waves accompanied by storm surge were identified as the principal cause of damage at the inlet. Unfortunately, no reliable wave data existed for the site. Therefore, the design wave height was based on depth-limited breaking wave height criteria. The design depth of water at the seaward end of the jetties was calculated to be 29 ft, based on a MLW depth at the structure toe of 18 ft, plus 5.5 ft maximum spring tide height, plus 5.5 ft storm surge elevation. Using procedures from the *Shore Protection Manual* (SPM) (1984) for a range of wave periods, T, from 7 to 15 sec and assuming a nearshore bottom slope of 0.01, values of the breaking wave height, H_b, ranged from 23.5 ft for 9-sec waves to 24.7 ft for 13 sec or longer wave periods. The design breaking wave height selected was, therefore, 25 ft.

Several alternative designs were considered for the rehabilitation, including 12- and 20-ton stone and 16-ton dolosse. Dolosse were found to have the lowest annual cost and were chosen for construction. A decision was made, based on engineering judgment, to reinforce the dolosse with epoxy-coated reinforcing rods (Figure 6).

Summary of Dolos Use

Dolos armor units were invented by Eric M. Merrifield, a South African engineer, in 1963 and first used by him in 1964. Initial model tests of the unit were performed by the South African Council for Scientific and Industrial Research in 1965 and showed that dolosse had a stability coefficient significantly higher than any other armor unit (Merrifield 1974). Subsequently, other laboratories tested dolosse and verified that they were more stable than other units. There has been some controversy over the actual magnitude of the stability coefficient since the units were first tested, resulting in different coefficients being published by various researchers. The coefficient has even been varied among different versions of the same design manual.

Because of their exceptional stability, dolosse have been used on a variety of projects throughout the world. In 1981, Zwamborn and Niekerk (1981) reported that 48 projects using dolosse had been built, were being constructed, or were under design. Their list omitted the jetties at Manasquan Inlet and the breakwater rehabilitation at Cleveland, OH, so the number is at least 50.

Between 1963 and the present, many lessons have been learned about dolosse, and some have become cautious concerning their use. Spectacular failures, such as the one at Sines, Portugal, in 1978, and the subsequent analysis of the events have revealed shortcomings in the techniques used to model test large structures in deep water. The problem is not associated with dolosse alone, but involves the use of any armor on deepwater structures where the effect of the larger waves on the survivability of the armor units is not well understood.

Reinforcement of dolosse is a rarity. In South Africa, only some of the dolosse used in Merrifield's original structure were reinforced, and those with only a single piece of scrap rail inserted in the center of the unit. Outside South Africa, reinforcement was first tried at Humboldt Bay, CA. Subsequently, as reported by Zwamborn and Niekerk (1981), reinforced units were used at Kahului, HI; in Japan; and in Namibia. Most recently, fiber reinforcement was used in new dolosse placed on the Crescent City, CA, breakwater. Unfortunately, little has been reported to date on the advantages or disadvantages of reinforcing the units, so the decision to reinforce is often left to the judgment of those involved with the structure design, as was the case at Manasquan Inlet. Data collection at Crescent City has shown that for large dolosse, most, if not all, of the tensile strength may be used in supporting other units stacked on lower dolosse. It should be noted that the objective of the use of reinforcement may vary just as the type of reinforcement varies. Fiber reinforcement, for example, is most often used for improved durability rather than increased tensile strength.

Monitoring Completed Coastal Projects Program

The jetty rehabilitation project at Manasquan Inlet was selected for monitoring under the Monitoring Completed Coastal Projects (MCCP) Program in 1982 during the program's second year. The program has as its goal the advancement of coastal engineering technology. It is designed to determine how well projects are accomplishing their purposes and are resisting the attacks of the physical environment. Those determinations, combined with concepts and understanding already available, will lead to upgrading the credibility of predictions of cost effectiveness of engineering solutions to coastal problems; to strengthening and improving design criteria and methodology; to improving construction practices; and to improving operation and maintenance techniques. Additionally, the monitoring program will identify concerns that laboratories should address more intently. Stated in another way, the objective is the advancement of the engineering science derived from insights into the physics that laboratory studies have developed.

To develop the direction for the MCCP Program, the Corps of Engineers (CE) established an Ad Hoc Committee of coastal engineers and scientists This committee formulated the program's objectives, developed its operational philosophy, recommended funding levels, and established criteria and procedures for project selection. A significant result of their efforts was a prioritized listing of problem areas to be addressed, essentially a listing of the program's areas of interest (Table 1). The initial list compiled had only the first 20 items. As the program has grown, so has the list; the final three items were recently added.

The selection process has worked well since the first projects were nominated in 1981. Periodically, the CE coastal offices are invited to nominate projects for monitoring under the program. Nominations are reviewed and prioritized by a selection committee composed of representatives from Headquarters, US Army Corps of Engineers (HQUSACE), the Coastal Engineering Research Center (CERC), US Army Engineer Waterways Experiment Station (WES); and the CE coastal Division offices. Final selection is based on the prioritized list of projects and available funding.

While guidance is provided by HQUSACE, management of the program rests with CERC. Operation of the program is a cooperative effort between CERC and the individual CE District offices. Development of the monitoring plan and conduct of data collection depend on the combined resources of CERC and the Districts.

Table 1
MCCP Program Areas of Interest

1.	Shoreline and nearshore current response to coastal structures.
2.	Wave transmission by overtopping.
3.	Prediction of controlling cross section at inlet navigation channels.
4.	Wave attenuation by breakwaters (submerged and floating).
5.	Bypassing at jettied and unjettied inlets.
6.	Wave refraction and steepening by currents.
7.	Beach-fill project monitoring.
8.	Stability of rubble structures - investigations to determine causes of failure.
9.	Comparison of pre- and postconstruction sediment budgets.
10.	Wave and current effects on navigation.
11.	Dynamics of floating structures.
12.	Wave reflection.
13.	Effects of construction techniques on scour and deposition near coastal structures.
14.	Diffraction around prototype structures.
15.	Wave runup on structures.
16.	Onshore/offshore sediment movement near coastal structures.
17.	Harbor oscillations.
18.	Wave transmission through structures.
19.	Material life cycle.
20.	Ice effects on structures and beaches.
21.	Model study verification.
22.	Wave translation.
23.	Construction techniques.

2 Monitoring Program

The single most important criterion for evaluating the Manasquan jetty rehabilitation is the structural stability of the jetties and, in particular, the dolos armor units. The relative success of the rehabilitation can be judged on how well the armor units resist displacement and breakage and protect the jetty underlayers during periods of storm wave attack. For this reason, a principal objective of this monitoring program was to establish the base conditions and then monitor any potential changes in the location and orientation of the dolosse. It was also necessary to monitor wave conditions at the site to allow correlation between any dolos movements and incident wave height. It was understood that any gross displacement of dolosse that exposed the stone underlayers would indicate a lack of success.

Two secondary but related areas of concern were evaluated in judging the relative success of the jetty rehabilitation. The first was the ability to maintain a stable inlet channel cross section. The second was whether the rehabilitated jetties caused adverse effects on adjacent beaches. It should be noted that neither of these concerns was considered a primary factor in justifying jetty rehabilitation. However, the potential does exist for the jetty rehabilitation to affect the navigation channel and nearby beaches. Regarding the inlet cross section, it was expected that stable jetty structures would enhance channel stability, as sand is not able to pass through the jetties and deposit in the channel, nor would unstable stone or dolosse be displaced from the jetty head or trunk into the channel. The jetties form a partial littoral barrier to the net northward longshore sediment transport, resulting in a discontinuity in shore alignment, with a net accretion to the south of the south jetty. It was expected that rehabilitation of the jetties would not significantly affect the shore alignment. Figures 7 and 8 show that expectation to be reasonable, as there is no detectable change in alignment on the adjacent beaches.

The monitoring program provided a means to document and quantify potential effects related to jetty rehabilitation on the inlet channel and adjacent beaches. The following section of this report presents the actual tasks accomplished under the monitoring program. These tasks provided the necessary data to evaluate the level of success of the jetty rehabilitation project in meeting the three objectives.

Data Collection

The three objectives of the monitoring effort were to evaluate the performance of the dolos armor units in maintaining the structural stability of the jetties, determine the potential effects of the rehabilitated jetties on longshore sediment transport in the vicinity of the inlet, and establish the effectiveness of the rehabilitated jetties in maintaining a stable inlet channel cross section. Table 2 presents a list of those tasks performed as a part of the monitoring program and identifies the objectives addressed by each task.

Table 2
Monitoring Program Tasks

Tasks	Monitoring Objective Addressed		
	Jetty Structural Stability	Shoreline Response	Inlet Stability
1. Leo[1]	X	X	X
2. Wave gage	X	X	
3. Tide gage	X	X	X
4. Tidal prism	X		X
5. Side scan survey	X		X
6. Inlet hydro survey			X
7. Beach surveys (on/offshore)		X	X
8. Aerial photography	X	X	X
9. Dolos stability–surveys	X		X
10. Dolos stability–photogrammetry	X		X
11. Site inspection	X	X	X
12. Project management	X	X	X
13. Data reduction/reports	X	X	X

[1] LEO = Littoral Environmental Observations.

Littoral Environment Observations (LEO, Task 1) were obtained twice daily from the seaward end of the south jetty at Manasquan Inlet between 8 June 1982 and 15 October 1984. The observations included wind speed and direction; breaking wave height, period, angle of approach, and type; surf zone width; and presence or absence of beach cusps and rip currents. The LEO measurements provided the only wave data during several periods when the wave gage was inoperative. The LEO observer, Donald R. Geberts, attained a 98-percent completion rate for making twice daily observations during the 28-month period of record.

A Datawell 0.7-m-diam Waverider wave measuring buoy (Task 2) was deployed on 9 September 1982 in 50 ft of water about 1 mile northeast of

the seaward end of the north jetty. This deployment was completed with the assistance of a boat and crew provided by the New Jersey Department of Environmental Protection, Bureau of Coastal Engineering. The buoy telemetered wave height and period data to a receiving station located in a US Coast Guard (USCG) facility in Manasquan, NJ. The data were regularly transmitted by phone line to the CERC Field Research Facility at Duck, NC, where they were processed. The gage performed satisfactorily until about 24 October 1982 when a severe coastal storm began to impact the entire mid-Atlantic coastline. The gage was lost during that storm. Interestingly, some months later a wave buoy was sighted at sea by an American flag freighter. Subsequently, the buoy was found by a fisherman in the Azores, turned over to US Air Force officials there, and returned to CERC over 2 years later still in good condition. A replacement gage was obtained and deployed on 30 November 1982. That gage operated satisfactorily until about 1 February 1983 when faulty data began to appear in the record. It was repaired and redeployed on 15 March 1983, having suffered an apparent collision with a vessel. The gage then operated satisfactorily until a lightning storm on or about 25 July 1983 disabled the shore station that received and recorded the raw wave data telemetered from the buoy. This problem was not corrected until 10 December 1983. After this repair, the gage operated successfully with only routine maintenance until its removal on 16 September 1985. Data were collected throughout the severe storm of 28-30 March 1984.

A recording stilling well tide gage (Task 3) was installed by the National Oceanographic and Atmospheric Administration (NOAA), National Ocean Service (NOS), for the monitoring program on 2 August 1982 at the USCG station in Point Pleasant Beach, NJ, and was operated continuously until its removal on 28 November 1984. Data from the period of record have been reduced by NOS.

Field measurements of tidal current velocity, tide height, and inlet cross-sectional area (Task 4) were made by contractors on 16 September 1982 and 9 August 1983 during spring tide conditions. Current measurements were made twice hourly at three depths at each of three locations across the throat of the inlet for a 13-hr period.

The initial side-scan sonar survey (Task 5) of the structures was performed on 29-30 September 1982. Results of that effort were only marginally useful. A second attempt to apply side-scan sonar techniques at Manasquan Inlet was made on 20 July 1984. A significant improvement in image quality was achieved relative to the 1982 attempt. However, heavy boating traffic through the inlet reduced the effectiveness of the system in resolving the underwater configuration of dolosse and armor stone because of the air entrained in the water column.

Inlet hydrographic surveys (Task 6) were obtained under the MCCP Program on the following dates: 4-5 October 1982, 9 September 1983, 3 October 1983, 4-17 January 1984, and 8-10 April 1984. The area covered in

these surveys included a zone extending from about 1,000 ft offshore of the seaward ends of the jetties, and then through the jettied and bulkheaded inlet channel to about 1,600 ft west of the inshore end of the inlet channel. The total length of this surveyed zone was about 5,200 ft, and ranged from about 400 ft wide in the jettied channel to about 1,000 ft wide offshore of the jetties. An additional survey was accomplished in September 1984 under the USAED, Philadelphia, Operations and Maintenance Program. These data have also been reviewed.

Survey baselines were established along the seawardmost streets of Manasquan and Point Pleasant Beach, NJ, the towns north and south of the inlet, respectively, in September 1982. Seventeen cross-section lines (Task 7) were established generally perpendicular to the baselines, with nine in Manasquan and eight in Point Pleasant Beach. Each baseline extends for a distance of about 5,300 ft from the inlet. Figure 9 shows the location and orientation of each cross-section line with respect to the inlet and adjacent beaches.

The onshore portion of each cross section uniformly begins at the baseline point and extends out to wading depth for a land survey crew, typically about -1 to -3 ft MLW. The offshore portion of each cross section extends inshore as far as practicable into the surf zone and offshore at least to a depth of -30 ft MLW, which is encountered on the order of 1,800 to 2,000 ft seaward of the baseline. The dates of onshore and offshore survey coverage are summarized in Table 3.

Table 3
Onshore/Offshore Survey Dates

Manasquan		Point Pleasant Beach	
Onshore	**Offshore**	**Onshore**	**Offshore**
30 Sep 82	—	29 Sep 82	—
3 Nov 82	—	4 Nov 82	—
11 Mar 83	—	10 Mar 83	—
8 Jun 83	—	7 Jun 83	—
16 Aug 83	1-2 Sep 83	15 Aug 83	7 Sep 83
20-21 Sep 83	20-21 Sep 83	22 Sep 83	22 Sep 83
28 Dec 83	6 Jan 84	29 Dec 83	18 Jan 84
26 Mar 84	—	27 Mar 84	—
3 Apr 84	12 Apr 84	2 Apr 84	11-12 Apr 84

The aerial photography (Task 8) obtained in the monitoring program includes periodic shore-parallel flights covering a 20-mile-long zone centered on Manasquan Inlet, combined with lower altitude flight lines over the inlet jetties. The former, with a contact scale of 1:4,800 (1 in. = 400 ft), are used to assess general shoreline conditions in the study region, whereas the latter,

with a contact scale of 1:1,200 (1 in. = 100 ft), are used as the database for photogrammetric mapping of the dolosse on the jetties. Combined high- and low-altitude flights were obtained in January, June, and September 1983 and in March, May, June, and October 1984 as part of the MCCP Program funded monitoring activities. An additional set of aerial photographic data was obtained in May 1985 with USAED, Philadelphia, funds.

In view of the historic problems with maintaining the integrity of the quarrystone armor layers of the jetties and the relative lack of prototype experience with dolosse on the US east coast, a technique was needed to accurately monitor the performance of the dolosse at Manasquan Inlet (Task 9) and to verify the validity of the design procedures and assumptions used in the rehabilitation effort. Therefore, as an experiment in the Manasquan Inlet monitoring program, the use of precision photogrammetry was proposed as a means to answer the following questions:

a. Do the dolosse move, particularly under storm conditions?

b. If they move, then how far and at which locations on the jetties?

c. Do dolos movements compromise the predicted project performance?

d. How accurate are the photogrammetric measurements, and what is the resolution (vertical and horizontal) of photogrammetry in this application?

e. Was jetty rehabilitation with dolosse successful?

f. Is photogrammetry a cost-effective method of monitoring the stability and performance of armor units on coastal structures?

The initial step in constructing photogrammetric maps (Task 10) of the south and north jetties at Manasquan Inlet was to establish primary targets on stable portions of the jetties and adjacent land area. The targets were surveyed in from nearby geodetic and vertical control benchmarks and were visible in the aerial photography. These primary targets were used to define the horizontal (x and y) and vertical (z) datums to which all measurements on the dolosse were referred. The primary targets on the jetties were located along the center line of the concrete cap. Each concrete cap section is a monolith 20 ft square by 6 ft thick, weighing 180 tons, and supported by the core material of the jetty. Primary targets were surveyed periodically to determine their stability as reference points.

Vertical black and white aerial photography was obtained with a shore-parallel flight line at an altitude of 600 ft, resulting in a contact scale of 1:1,200. The photographic flights were scheduled to coincide with times of low tide and high sun angle. All photography in this monitoring program was obtained with the same precision cartographic camera, a Zeiss

RMK A 15/23. A total of three exposures were required to prepare the photogrammetric maps of the jetties; the southern and middle exposures were used for the stereo model of the south jetty, and the middle and northern exposures were used for the stereo model of the north jetty.

The final step in constructing the photogrammetric maps was compilation. A Kern PG 2-AT stereo restitution instrument was used to compile the selected features, in this case the plan view outlines of the dolosse, concrete cap sections, and armor stone. These features were superimposed on a grid based on the New Jersey State Plane Coordinate System, which graphically defined location and orientation of features in the horizontal plane. Vertical data were recorded numerically as spot elevations at selected points on the same features. Using an enlargement factor of 20 times the contact scale, the finished scale of the maps was 1:60 (1 in. = 5 ft). A portion of the south jetty map is shown in Figure 10.

The photogrammetric maps were plotted on stable-based transparent drafting material. In this manner, the stability of a dolos from one flight date to the next was determined by overlaying and registering the two maps, then visually comparing the location of the feature of interest on the earlier and later dates. If a dolos moved in the time interval, the horizontal component of movement was evident as a displacement of the outline of the dolos, which was then scaled from the 1:60-scale maps. Experience with a number of Manasquan Inlet jetty maps has shown that horizontal movements of as little as 0.3 ft can be reliably detected. The vertical component of movement was determined by comparison of the spot elevation data for a particular point. A later section of this report addresses the accuracy and resolution of the vertical measurements.

Table 4 details the dates of aerial photography that were suitable for photogrammetric mapping. Also shown are the dates of the three most significant storm events that occurred during the course of the Manasquan Inlet monitoring program. The symbols "S" and "N" refer to the south and north jetties respectively, and the numbers (1, 2, etc.) refer to successive photogrammetric maps prepared for each jetty. Note that a north

Table 4
Dates of Aerial Photography and Storm Events

Date	Event	South Jetty	North Jetty
9 Jan 82	Photo	S1	—
24-26 Oct 82	Storm		
29 Jan 83	Photo	S2	N1
12-13 Feb 83	Storm		
15 Sep 83	Photo	S3	N2
27 Mar 84	Photo	S4	N3
28-30 Mar 84	Storm		
9 May 84	Photo	S5	N4

jetty map was not prepared from the 9 January 1982 photography because the north jetty rehabilitation was in progress at that time.

The initial maps for the south jetty (S1) and the north jetty (N1) are the most detailed of the maps prepared in this monitoring program. Spot elevations were determined at two or three locations on all fully visible dolosse and cap sections, with one or two elevations determined for partially visible dolosse and armor stones. Together, these two maps document the location, orientation, and elevation of 754 dolosse, about 57 percent of the 1,326 units placed on the two jetties during the 1979 to 1982 rehabilitation. The remaining 43 percent of the dolosse were not mapped because they were either under water or beneath the top layer of dolosse and, thus, not visible in the photography. Subsequent photogrammetric maps have typically included from 20 to 30 percent of the 754 dolosse shown on the initial maps. This smaller sample size reduces the cost of map compilation while still obtaining representative coverage of armor units on the two jetties.

As a check of the accuracy of photogrammetric methods that were previously untested in mapping armor units, standard leveling techniques were used to record two or three spot elevations on a representative sample of both dolosse and armor stones. These level data were obtained from 65 south jetty dolosse on 27 April 1982, 14 March 1983, and 8 September 1983, and for 95 north jetty dolosse on 15 March, 9 June, and 7 September 1983.

Data Analysis

The LEO data were submitted monthly to CERC for coding into computer files. The data were analyzed by CERC, yielding the statistics shown in Table 5. Figure 11 is an example of the result.

Table 5
Summary of LEO Derived Statistics[1]

Breaking Wave Heights		Breaking Wave Periods		Breaking Wave Types	
Heights, ft	**Percent**	**Periods, sec**	**Percent**	**Types**	**Percent**
0-1.9	36	0-5.9	10	Spilling	37
2.0-3.9	42	6.0-7.9	35	Spill-plunge	37
4.0-5.9	17	8.0-9.9	36	Plunging	14
6.0-7.9	4	10.0-11.9	16	Surging	12
8.0-9.9	1	>12	3	Calm	~0
>10					

[1] Mean breaking wave height: 2.8 ft.
Mean wave period: 8.1 sec.

A review of the LEO data shows that during 8 months each year, waves approaching from the right (southeast quadrant) predominated, while during the 4 months of October, January, February, and March, waves from the left (northeast quadrant) dominated. This finding is qualitatively consistent with historic evidence that the net longshore transport direction at Manasquan Inlet is from south to north.

A comparison was made between the visual observations and the wave gage data. Before discussing this comparison, though, a caution is appropriate. The LEO were taken at the head of the south jetty in about 20 ft of water. The buoy was in about 50 ft of water a mile north of the inlet. The LEO data set was more complete than the gage data set, the observer achieving nearly a 98-percent completion rate for his twice daily observations. It can be expected that the LEO database would contain a wider range of conditions, such as sometimes breaking waves at the jetty head, than the gage record, since the LEO record is more nearly continuous. Table 6 shows a comparison of LEO data to gage results for each total record. The percent occurrence of waves in certain height ranges for ranges of periods is shown. Recognizing the differences in the data sets, a few general observations can be made. First, the LEO data agree more closely with the gage data on wave period than on height. That is not surprising since wave period changes relatively little as the wave refracts and shoals and can be measured using a watch whereas height has to be estimated visually. One surprise is that the observer estimated lower waves than the gage measured. Since the waves would shoal at the structure, one would expect the waves to be typically higher there than farther offshore. The LEO record indicates that 95 percent of all waves have a significant wave height of less than 6 ft. The gage record indicates this height is 10 ft, a considerable difference. In general, the LEO and wave data agree best with waves of moderate height and period. An example from a storm of late March/early April 1984 is shown in Figure 12. The LEO data seem best correlated to the gage when wave heights are under 2 m and periods around 6 to 8 sec. Again, difference in the location of the data sets must be emphasized. It is certainly part of the cause of the difference in height in particular.

A study by Douglass and Weggel[1] used the LEO data set to estimate the longshore transport and, therefore, bypassing requirements at Manasquan. That study was prepared for USAED, Philadelphia, and actually used both LEO and Wave Information Study (WIS) Phase III hindcast wave data (Jensen 1983a, 1983b) to prepare the transport estimate. The use of WIS data allows the comparison with several other reports, including a recent report by Gravens, Scheffner, and Hubertz (1989); Douglass (1985); and earlier work in the area by Caldwell (1966) and Fairchild (1966). Comparison of the results of these studies can provide

[1] S. Douglass and J. Weggel, 1986, "Estimation and Synthetic Gerneration of Longshore and Transport Data and Simulation of Sand Bypassing at Manasquan Inlet, New Jersey," unpublished contract report, USAED, Philadelphia; Philadelphia, PA.

Table 6
LEO Data Versus Gage Results

Period, sec	Percent Occurrence, Wave Height, ft										Total Percent	Cumulative Percent
	0-1.9	2-2.9	3-3.9	4-4.9	5-5.9	6-6.9	7-7.9	8-8.9	9-9.9	>10		
LEO Data												
0-1.9	0	0	0	0	0	0	0	0	0	0	0	0
2-3.9	0.1	0.1	0.1	0	0	0	0	0	0	0	0.3	0.3
4-5.9	3.6	3.3	1.9	0.4	0.3	0.2	0	0	0.1	0	35.1	10.1
6-7.9	12.2	8.9	6.0	4.5	2.2	0.7	0.2	0.2	0.1	0.1	35.9	45.2
8-9.9	13.4	8.9	6.5	3.1	2.2	1.0	0.4	0.1	0.2	0.1	35.9	81.1
10-11.9	5.6	3.0	2.4	1.8	1.7	1.0	0.3	0.1	0.1	0	16.0	97.1
12-13.9	0.6	0.5	0.5	0.5	0.2	0.1	0	0	0	0	2.4	99.5
14-15.9	0.5	0	0	0	0	0	0	0	0	0	0.5	100.0
>16	0	0	0	0	0	0	0	0	0	0	0	100.0
Total percent	36.0	24.7	17.4	10.3	6.6	3.0	0.9	0.4	0.5	0.2		
Cumulative percent	36.0	60.7	78.1	88.4	95.0	98.0	98.9	99.3	99.8	100.0		
Gage Results												
0-1.9	0	0	0	0	0	0	0	0	0	0	0	0
2-3.9	1.8	3.6	0.7	0	0	0.3	0	0	0	0	6.4	6.4
4-5.9	1.9	5.9	4.7	3.3	1.1	1.0	0.2	0.2	0	0	18.3	24.7
6-7.9	2.7	5.8	3.0	2.0	2.6	0	0.3	0.3	0	1.3	18.0	42.7
8-9.9	12.2	11.2	5.7	2.9	1.1	1.0	1.1	0.7	1.6	4.0	41.5	84.2
10-11.9	2.1	3.8	1.2	1.0	1.5	0	0	0	0	0	9.6	93.8
12-13.9	0	1.0	3.0	0.2	0	0	0	0	0	0	4.2	98.0
14-15.9	0	0	1.4	0	0	0	0	0	0	0	1.4	99.4
>16	0	0	0.3	0	0.3	0	0	0	0	0	0.6	100.0
Total percent	20.7	31.3	20.0	9.4	6.6	2.3	1.6	1.2	1.6	5.3		
Cumulative percent	20.7	52.0	72.0	81.4	88.0	90.3	91.9	93.1	94.7	100.0		

some guidance on the applicability of various data sets to longshore transport calculations.

Douglass and Weggel[1] used the time series of local height, period, and direction of sea and swell from the 20-year (1956-1975) WIS data set to calculate longshore transport rates. WIS Phase III Sta 56, located approximately 3 miles south of the inlet (Figure 13), was used for the analysis. One of the 20 years (1975) of WIS wave data was discounted because of unrealistically constant wave heights. The remaining 19 years produced annual sand transport values shown in Table 7. Although the variability of the annual net transport rate seems large, it is smaller than that of other east coast open ocean sites that have been studied.

Table 7
Net Annual Longshore Sand Transport Rate as Calculated from WIS Hindcast Wave Data (Negative to the North)

Year	Net Transport Rate cubic yards/year
1956	41,000
1957	-299,000
1958	-114,000
1959	-441,000
1960	-370,000
1961	-202,000
1962	6,000
1963	-299,000
1964	-294,000
1965	-263,000
1966	-233,000
1967	-210,000
1968	-329,000
1969	-160,000
1970	-333,000
1971	-344,000
1972	-353,000
1973	-794,000
1974	-522,000

Using WIS wave data, the net annual longshore transport resulting from the study was 280,000 cu yd northward. Because the calculated transport was larger than anticipated, the LEO wave data collected during this monitoring effort were used to calculate transport rates. Those are shown in some detail in Table 8. Table 9 provides a variety of transport rates calculated for the study area. The difference between the transport resulting from LEO wave data and WIS wave data is large (Figure 14). Of all the studies, only LEO data produce a southerly net transport. When the results obtained by Douglass and Weggel[1] are compared with the transport calculated as a part of the LEO output (Figure 15), there is relatively good agreement. Using LEO wave data, Douglass and Weggel calculated a net southerly transport of 600,000 cu yd, compared with 656,000 cu yd calculated by the LEO program. The remaining calculations are also comparable. This indicates that the errors in direction and magnitude of the net transport may be caused by problems with the basic measurements. There are several possible reasons for the disagreement. First, the LEO data were obtained at the head of the south jetty. Conditions there can be expected to be quite different from nearer to the shore in the zone of principal sediment transport. Second, as with the gage data, the WIS information was derived for a location some distance from the LEO site. Differences can be anticipated. Last, and possibly most important, the directions produced by the different techniques might be expected

[1] Op. cit.

to be different. In fact, Douglass and Weggel[1] noted that the direction and magnitude of the two data sets are quite different. They commented that the LEO data might be biased toward small waves since they were taken during three summers but only two winters. Another possible contributor to the differences is the bathymetry. The bottom near the head of the jetty is quite complex. Bathymetry at the WIS Phase III site is assumed to be regular. The same wave transformed over both bottoms could look quite different, so the observer may have seen a wave whose crest had quite a different orientation than that predicted by WIS. Wave directions at the

Table 8
Transport Volumes Estimated from Leo (thousands of cu yd)

Month	Southward Transport	Northward Transport	Net Transport	Gross Transport
Jun 1962	68	-123	-55	191
Jul 1982	4	-75	-71	79
Aug 1982	27	-60	-33	97
Sep 1982	163	-72	90	235
Oct 1982	280	-47	234	327
Nov 1982	104	-128	-24	232
Dec 1982	103	-67	36	170
Jan 1983	128	-24	104	152
Feb 1983	421	-4	417	425
Mar 1983	351	-51	300	402
Apr 1983	113	-156	-44	269
Jun 1983	22	-36	-14	58
Jul 1983	4	-30	-27	34
Aug 1983	23	-44	-22	67
Sep 1983	116	-21	94	137
Oct 1983	158	-22	135	180
Nov 1983	42	-62	-20	104
Dec 1983	113	-63	50	176
Jan 1984	176	-31	145	207
Feb 1984	44	-37	7	81
Mar 1984	131	-26	105	157
Apr 1984	77	-31	45	108
May 1984	9	-72	-64	81
Jun 1984	8	-37	-29	45
Jul 1984	4	-71	-68	75
Aug 1984	29	-17	12	46
Sep 1984	69	-21	48	90
Oct 1984	99	-38	60	137

[1] Op. cit.

Table 9
Summary of Longshore Transport Rates Developed for Study Area

Reference	Transport (in cubic yards per year)				Comments
	North	South	Gross	Net	
PRC Harris, Inc., 1980, "Special Study of Sand Bypassing, Manasquan Inlet, NJ," unpublished report prepared for USAED, Philadelphia	1,800,000	1,500,000	3,300,000	300,000 N	Energy flux, SPM method; input wave data from Little Egg Inlet vic. wave gage. Other lower values reviewed, discounted.
Douglass & Weggel, op. cit.	500,000	220,000	720,000	280,000 N	Energy flux, SPM method, input wave data from WIS Phase III data tapes, Sta 56.
University of Delaware (1980)	500,000	348,000	848,000	152,000 N	Recon-level report. Assumed constant 500,000 QNorth along north Jersey coast, and QNet = 0 at Dover Township, QNet = 500,000 north at Sandy Hook, linear interp. for Manasquan Inlet Location. Rejects PRC Report.
N. C. Kraus, M. Gravens, and D. Mark, 1986, "Coastal Processes, Sea Bright to Ocean, Township, NJ," draft report prepared by WES, Vicksburg, MS, for USAED, New York	—	—	—	61,000 N[1] (121,000 N)	Used Phase II WIS Data (Sta 23) in to 60 ft, then fine mesh grid to shoreline. Values shown as QNet are for Shark R. Inlet S. limit of transport calculations.
USAED, New York (1954)[2]	—	—	—	74,000 N	This report presents coastal erosion rate measurements assumed to represent net trans. rates. Same values as Caldwell (1966).
S. C. Farrell, 1981, "Evaluation of Longshore Sand Transport Manasquan Inlet, New Jersey," unpublished report prepared for USAED, Philadelphia	—	—	—	45,000 N	Used 1965-77 aerial photo and channel survey data—beach & channel volume changes assumed to equal net transport rate.
Bruun (1978)	—	—	70,000 to 110,000	—	Approach says inlet geomorphology and "behavior" represented by ratio of Tidal Prism/Gross Littoral Transport Rate (SL/MTOT) at Manasquan, know Ω and Ω/M ratio, calc. MTOT.
USAED, Philadelphia (1986)	—	—	—	—	1961-1986 shoaling rate (inlets sediment trapping rate) is 20,500 cu yd/year. No transport rate implied.

1 Value in parentheses based on "constant" rather than "variable" maximum depth of closure.
2 Report became HD 361 (84-2) March 1956.

jetty tip may also be influenced by flood- and ebb-tidal currents. One final aspect of this difference in angles is that the LEO observer reports only the dominant wave train, even though several wave trains may be present. This emphasis of the visually dominant wave train can bias the LEO. In all, there seems to be reason to apply the transport rates produced by the LEO program with caution.

As might be expected, there is some variation in the results of different studies done in the Manasquan area. Table 9 lists some of the studies done in the past. The *P.R.C. Harris* study appears to be unrepresentative of the conditions at Manasquan Inlet, possibly because they relied on wave data from Little Egg Inlet, some 45 miles to the south and nearly 35 miles south of a generally accepted nodal zone. It might be noted that the use of WIS shallow-water wave data produced a northerly net transport rate higher than all but the *P.R.C. Harris* study.

Seasonal trends appear in the 19 years of data in the study by Douglass and Weggel[1]; Table 10 and Figure 16 illustrate those changes. For some portion of the time, waves are smaller than the threshold value for the WIS model and no transport results, explaining the failure of Figure 17 to produce monthly percentages that sum to 100 percent.

A more recent study (Gravens, Scheffner, and Hubertz 1989) also used WIS data from Sta 56 to develop transport rates for the region around Manasquan Inlet. Their work was more detailed than that of Douglass and Weggel,[1] beginning with the same numerically developed wave data, then adding bathymetry to their model and refracting the waves to shore. Interestingly, their results were quite similar to those of Caldwell (1966), results that were reaffirmed and discussed by Fairchild (1966) and Ashley, Halsey, and Buteux (1986). In the earliest study, Caldwell (1966) estimated that the gross transport along the New Jersey shore was approximately 500,000 cubic yards/year with a net northerly transport to the north of a nodal point between Manasquan Inlet and Barnegat Inlet. Caldwell estimated the net transport in the vicinity of Manasquan Inlet to be 74,000 cubic yards/year, while Gravens, Scheffner, and Hubertz (1989) estimated it to be 72,000 cubic yards/year (Figure 18).

As was discussed on pages 8 and 9, wave data collection was problematic until December 1983. After that time, data collection was continuous through September 1985 when the buoy was removed. Wave data were collected to correlate the response of the structure and shoreline to severe storms. During the last few days of March 1984, data were collected during a severe northeaster, which was a near-design storm. An evaluation of wave height versus period for the data collected during the monitoring effort were determined. The maximum H_s recorded during the storm, 21.8 ft, was seen during a data collection in the early hours of 29 March. During the course

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1 Op. cit.

Table 10
Monthly Net Longshore Transport Rates (thousands of cu yd)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1956	1050	750[1]	83	-638	-98	-83	-623	-45	1103	668	128	-135
1957	45	255	720	1035	-150	-270	-120	-45	-195	-323	-960	-1418
1958	248	-720	1283	1005	-255	-210	-113	-308	-120	60	-368	638
1959	930	-628	-810	30	-38	-45	-383	-98	-158	-1035	-810	-375
1960	-45	-1950	-563	-728	-465	-660	-225	8	-8	443	-15	-233
1961	300	-300	-285	908	-705	-735	323	-225	690	743	-300	-383
1962	-405	-38	2213	-780	-98	-90	-75	-30	83	-8	-255	-443
1963	-413	-510	-885	-368	-990	-668	-150	-248	1118	488	-683	-285
1964	915	-390	0	-578	480	-105	-158	-210	473	-83	-780	-1268
1965	-135	-908	-83	-248	-143	165	-240	-300	-135	-608	-98	-428
1966	518	-210	-255	-98	-158	-158	-225	-135	-368	-248	-608	180
1967	-300	-195	398	270	195	-38	-368	-720	-188	-495	-300	-780
1968	420	-45	-548	-390	-263	-615	-225	-173	-323	-180	23	-788
1969	105	968	-683	-225	-623	-135	-158	-98	413	203	-570	-1118
1970	23	-1703	-240	-270	-150	-218	-233	-38	-188	-248	-45	-683
1971	-263	-1463	-503	-195	-593	-128	-165	-653	-300	585	-233	-225
1972	-465	-293	-1643	-68	-660	-390	-23	-38	495	-90	-855	-203
1973	-623	-795	-353	-728	-698	-563	-135	-248	-113	-548	-863	-3870
1974	675	-645	975	-810	-578	-180	-345	60	-233	-68	-143	-1673

[1] Negative is to the north.

of the storm, data were collected as often as once an hour. During each data collection period, 17 min of data were obtained.

To compare the waves measured offshore during the March 1984 storm to the design wave, the **R**egional **C**oastal **P**rocesses **WAVE** (RCPWAVE) transformation model was used to simulate a variety of conditions measured during the storm. The grid used for this analysis is composed of constant-sized rectangular cells. It is oriented with the y-axis alongshore and the x-axis offshore. There are 45 cells alongshore, each 200 ft long. There are 50 cells in the offshore directions, each 100 ft long. Therefore, $dx = 100$ ft and $dy = 200$ ft.

Bottom contours in the area are generally straight and parallel to the coastline, except in the immediate vicinity of the inlet jetties. The depth grid generated for the RCPWAVE runs was constructed using several sources. From the shoreline to approximately the 30-ft-depth contours, hydrographic survey data from the 1984 CE beach and inlet surveys were used. Offshore from the 30-ft contour, NOAA and US Geological Survey sounding charts were used to estimate the location of the 40-, 50-, and 60-ft contour depths. Then between the contour lines, individual depths were interpolated for each grid cell. Each depth was referenced to mean low water at Manasquan Inlet and recorded in a data file for input into the model (Figure 19). The jetties are located in the model as follows:

a. South jetty: I1/J12 to I9/J10.

b. North Jetty: I1/J14 to I9/J12.

The wave data set collected during the March 1984 northeaster was used for this analysis. Three representative cases were chosen from the wave data for significant wave height and peak period combination:

Case 1: $H_s = 18.14$ ft, $T_p = 9.75$ sec.

Case 2: $H_s = 21.8$ ft, $T_p = 10.9$ sec.

Case 3: $H_s = 19.7$ ft, $T_p = 12.34$ sec.

The measured data do not include wave direction; therefore, for each of the three wave cases, three directions were run to see the effects of wave approach angle. The wave angles applied were approximately east, east-northeast, and northeast. The shoreline, and hence the grid, is oriented such that it makes an angle of 10 deg east of north. The wave data are given in 50 ft of water. With the version of RCPWAVE being used, it is not necessary to estimate the deepwater characteristics for input but just to specify the wave height, period, and direction in 50 ft of water.

The model can include the effects of the jetties on wave diffraction. So for each set of wave height period and direction, nine runs in all, a

simulation was made with and without the jetties. The RCPWAVE model output includes the wave height, approach angle, wave number, and breaking index at each grid cell. The results (output) of where the highest waves were measured were used as input for the case with jetties and an approach angle of 45 deg, as shown in Figures 20-23. The highest waves produced in the model were just over 22 ft offshore of the head of the jetties. Although the model shows this as a nonbreaking wave, the wave does break closer to the structure, supporting the use of a depth-limited, breaking wave in the design. The design was based on a 25-ft breaking wave, so while the March 1984 storm was not quite a design event, it was a good test of the structure.

Wave, water level, and profile data collected have been used subsequently in the verification of a model developed by Larson and Kraus (1989). Storm-Induced **BEACH** Change (SBEACH) model is intended to numerically simulate the response of fills and beach profiles to storms, as well as the initial response of fills to wave action. Data from the March 1984 northeaster were used for the verification. In particular, wading surveys on 27-28 March 1984 (prestorm) and completed surveys of 2-3 April 1984 were compared with the numerically generated profiles along both the Point Pleasant and Manasquan shores. Figures 24 and 25 show the water levels and wave height and period, respectively, during the storm. To develop a prestorm profile for each beach, a mean initial profile was developed using subaqueous data from 28 December 1983 and subaerial data from 27-28 March 1984. A mean profile was developed from survey data for the poststorm condition. The difference between the prestorm conditions at the two beaches is represented on Figures 26 and 27. For the model, a composite grain size was established from data from the 1950s and verified by samples for 1989, since no sediment samples were taken as a part of this monitoring effort. Recognizing that the beaches had experienced several days of recovery by the time of the poststorm surveys, the ability of the model to replicate change in the profile shape and predict the volume of erosion (Figures 26 and 27 and Table 11) was reasonably good. The fact that Point Pleasant experienced more erosion sug-

Table 11
Measured and Calculated Volumes of Profile Erosion (m^3/m)

	Measured				Calculated	
	Average of All	Average of Selected	Min	Max	Max Erosion	With Recovery
Point Pleasant Beach						
0-Contour	48.9	55.8	9.2	66.0	86.4	73.9
1-m Contour	45.8	53.8	8.5	63.1	54.6	54.2
Manasquan Beach						
0-Contour	36.3	39.3	14.7	62.6	51.1	44.2
1-m Contour	32.4	34.7	13.4	56.2	37.2	37.2

gests that the groins along the Manasquan shoreline may have had a stabilizing effect during the storm. One condition the model could not reproduce well was the large berm above mean sea level (MSL) at Point Pleasant. This caused an overestimation of the volume of eroded material on this beach. The shoreline at Manasquan did not experience similar berm recoveries, so the model estimates of erosion are in better agreement with the measurements.

Tidal data were obtained at the USCG station at Manasquan Inlet over the period August 1982 through November 1984. The data were collected and analyzed by the NOS and were provided to the USAED, Philadelphia, as monthly summaries in two formats: tabulations of time and height of high and low tides (Figure 28), and tabulations of hourly tide heights (Figure 29). At the conclusion of the tide observations, the entire time series was analyzed and tidal datums were determined for the tidal epoch 1960-1978 based on a comparison with long-term tidal data at Sandy Hook, NJ. Table 12 summarizes pertinent tidal data obtained from the Manasquan Inlet gage. Tidal datum elevations are referenced to local mean lower low water (MLLW), the datum adopted by NOS for displaying depths on navigation charts, and to National Geodetic Vertical Datum (NGVD), formerly known as Mean Sea Level of 1929.

Table 12
Tidal Datum Plane Elevations 1982 to 1984 Observation Period

Datum or Parameter	Elevation in Feet, MLLW	Elevation in Feet, NGVD
Highest observed water level (3/29/84)	8.41	6.99
MHHW[1]	4.51	3.09
MHW	4.18	2.76
MTL	2.18	0.76
NGVD	1.42	0.00
MLW	0.18	1.24
MLLW	0.00	-1.42
Lowest observed water level (1/21/84)	-1.88	-3.30

[1] MHHW = mean higher high water; MHW = mean high water; MTL = mean tide gage.

The tidal characteristics of Manasquan Inlet were previously monitored by the CE between 1933 and 1939 following construction of the jetties and initial channel dredging. Table 13 presents a comparison of various tidal statistics derived from the 1933-1939 and the 1982-1984 periods of observation. References to elevation in Table 13 use a common vertical datum, NGVD, for both periods.

The most significant changes evident in Table 13 are the increases in elevation of the tidal datum planes mean high water (MHW), mean tide level (MTL), and mean low water (MLW), which respectively rose 0.77, 0.72, and 0.67 ft. When these values are annualized over the 47-year

Table 13
Comparison of 1933-39 and 1982-84 Tide Statistics

Datum or Parameter	Units	1933-39 Value	1982-84 Value	Change, 1930s to 1980s
MHW	Feet, NGVD	1.99	2.76	+0.77
MTL	Feet, NGVD	0.04	0.76	+0.72
MLW	Feet, NGVD	-1.91	-1.24	+0.67
Mean range	Feet	3.90	4.00	+0.10
Highest obs.	Feet, NGVD	6.10	6.99	n/a
Lowest obs.	Feet, NGVD	-4.40	-3.30	n/a
GMHWI[1]	Hours	0.17	0.08	-0.09
GMLWI	Hours	6.34	6.22	-0.12
Duration of rise	Hours	6.25	6.28	0.03
Duration of fall	Hours	6.17	6.14	-0.03

1 GMHWI and GMLWI = Greenwich mean high- and low-water intervals, respectively.

period between midpoints of the two observation periods, the mean annual rate of rise of these datum planes is 0.016, 0.015, and 0.014 feet/year, respectively. These values are comparable to the long-term rate of sea level rise observed at other tide gage sites in this region. For example, at New York City, Sandy Hook, and Atlantic City, the long-term mean rate of sea level change over the period of record at each gage has been 0.011, 0.017, and 0.015 feet/year, respectively. Although the tidal datum planes have risen significantly at Manasquan Inlet between the 1930s and the 1980s, there has been a smaller increase (0.10 ft) in the mean tidal range. Similarly, there have been only relatively small changes in the Greenwich mean high- and low-water intervals (GMHWI and GMLWI) and in the durations of rise and fall of the tides.

The tide gage data from the March 1984 storm have also been analyzed. A plot of predicted and observed tide heights for the period 27 March through 31 March 1984 is presented in Figure 30. Figure 31 shows the same parameters but for only the 3 days of significant storm effects (28-30 March 1984). Figure 32 is a plot of the recorded surge height for the 28-30 March period, calculated as hourly differences between predicted and observed water levels.

In the development of the design for the jetty rehabilitation, it was determined that the design water level would be +11 ft MLW, assuming a 5.5-ft tidal height and a 5.5-ft storm surge. MLW is at -1.24 ft NGVD, so the design correlates to +9.76 ft NGVD. The maximum water level measured during the monitoring effort was +6.99 ft NGVD, so the storm surge approached but did not reach the design water level. Although the surge component was about 6 ft, the tide was falling at the time of the maximum surge.

In the recent history of Manasquan Inlet, there have been three significant storms that generated nearly equal maximum water levels: the

northeasters of 1962 and 1984, and Hurricane Gloria in 1985. The severity of the storms, as represented by the damages done, related most to the duration of the storm. As can be seen in Figure 33, the three storms had comparable maximum water levels. Hydrographs from the three storm events were superimposed (Figure 33) with the time of maximum surge height (not necessarily the same time as the maximum water level), defined as "hour 0." The +7 ft NGVD line is shown for reference to compare the maximum water levels of the three storms. It is the duration that is most dramatically different among the storms. The most damaging storm of the three was the northeaster of 1962, the storm of the longest duration. Hurricane Gloria, the storm of shortest duration, caused the least damage, as might be expected. Obviously, damage done by a storm involves much more than just the storm duration, but these data indicate that duration should be a serious consideration in design.

Two sets of tidal prism measurements were made at Manasquan Inlet during the course of the MCCP data collection. The dates of measurement were 16 September 1982 and 7 August 1983. Both dates were selected as representative of spring tidal conditions, which are considered to be the primary factor influencing tidal prism and minimum inlet cross-sectional area (Jarrett 1976).

A standardized measurement program was used on both dates, with three basic types of data obtained at a selected inlet cross section:

a. Current speeds and directions.

b. Tide height.

c. Hydrographic survey of the cross section.

Tides at Manasquan Inlet are semidiurnal; thus, data were collected for 13-hr periods in order to observe full tidal cycles.

Inlet Sta 18+00 was selected for both the 1982 and 1983 tidal prism measurements. This station is located approximately 1,800 ft landward of the head of the south jetty. The channel cross section at Sta 18+00 was larger on both dates than at the controlling inlet cross section located at approximately Sta 6+00. Measurements were made at Sta 18+00 rather than at Sta 6+00 because of the relatively lower degree of ocean wave exposure at the more landward location. The channel width between the vertical concrete bulkheads at Sta 18+00 was surveyed to be 401 ft.

Three current measurement stations were established at Sta 18+00 by anchoring mooring buoys at positions 1/6, 1/2, and 5/6 of the distance across the inlet. During the 13-hr periods of measurement, the three inlet stations were each occupied an average of two times per hour. At each station, current speed and direction were recorded at points 0.2, 0.5, and 0.8 of the total depth at the time. The current velocities were measured

with an ENDECO 110 ducted-impeller meter in 1982 and with a Marsh-McBirney 201 electromagnetic meter in 1983. Tide height at Sta 18+00 was recorded from a temporary staff at 15-min intervals to the nearest 0.1 ft. An individual sequence of flow measurements through the inlet cross section thus consisted of a discrete velocity at each of the nine unequal area subsections of the inlet (three depths times three locations). The sequence of current measurements was performed 27 times for each tidal prism observation.

The tidal prism volumes were calculated in the following manner. Each sequence of nine velocity measurements was assumed to have been made at the midpoint of the time interval of the sequence. The cross-sectional area of each of the nine subsections was computed based on the hydrographic survey data and the tide height at the midpoint of the time interval. The instantaneous discharge through each subsection was calculated as the product of the measured velocity and the subsection area. Flood velocities were treated as positive values and ebb velocities as negative values. The instantaneous discharge through the inlet was calculated as the sum of the nine subsection instantaneous discharges. The total discharge over a given time interval was calculated as the product of the inlet instantaneous discharge times the duration of the interval. The sum of the positive discharge values represented the flood-tidal prism, and the sum of the negative discharge values represented the ebb-tidal prism.

In the 1982 and 1983 tidal prism measurements, it was necessary to limit observations to daylight hours because of the navigational hazard of working at night in a narrow inlet with round-the-clock usage by recreational and commercial fishing vessels. It was therefore not possible to have the flow measurements begin and end at slack tide conditions. Both the 1982 and 1983 tidal prism observations began during inlet flood conditions approximately at the time of high tide. The following ebb portion of the tidal cycle was observed from slack high water to slack low water. This was followed by the second partial flood interval. As 13 hr of data were obtained on each measurement date, it was possible to synthesize a flood tidal prism value from the two less-than-complete flood intervals on each date by limiting flow calculations to a total elapsed time of 12.4 hr.

The values calculated for the 1982 and 1983 tidal prisms are presented in Table 14 along with other pertinent tidal hydraulic data obtained at the time of the measurements. Table 14 also presents tidal prism-related data obtained at Manasquan Inlet in 1931, at a time when the jetties were complete but the navigation channel had not been dredged to full width or depth, and in 1935, after the channel had been dredged to the then-authorized dimensions of 10 by 250 ft through the inlet. Present authorized dimensions are 14 by 250 ft.

At any given time, the tidal prism at Manasquan Inlet can be influenced by a number of factors other than ocean tide range. These factors include geometry of the inlet channel, rainfall in the basin tributary to Manasquan

Table 14
Tidal Prism Data

Date of Observation	Flow Direction	Observed Tidal Prism x10^8 ft^3	Tide Range Data, ft			Maximum Flow Velocity Cross-Section Avg.,[1] ft/sec	Cross-Section Area Below MSL, ft^2	
			Observed Inlet Throat	Observed USCG Station	Predicted Inlet		At Flow Observation Section	At Controlling Section
16 Sep 82[2]	Ebb	3.08	5.3 fall	4.9 fall	5.0 fall	-2.6	7,400	5,000
	Flood	3.80	6.1 rise	5.4 rise	5.5 rise	+3.0	7,400	5,000
9 Aug 83[2]	Ebb	3.08	5.6 fall	5.3 fall	5.2 fall	-3.2	6,200	5,200
	Flood	4.31	6.2 rise	6.2 rise	6.1 rise	+4.1	6,200	5,200
21 Oct 31[3]	Ebb	1.02	3.2 fall	—	3.2 fall	-2.5	2,200	2,200
	Flood	1.00	3.3 fall	—	3.4 rise	+2.7	2,200	2,200
22 Oct 31	Ebb	0.84	3.0 fall	—	3.8 fall	-2.2	2,200	2,200
	Flood	1.00	3.3 rise	—	3.9 rise	+3.2	2,200	2,200
12 Jun 35[3]	Flood	1.87	3.8 rise	—	4.2 rise	+2.3	4,600	4,600
	Ebb	1.61	3.7 fall	—	4.1 fall	-2.0	4,600	4,600
13 Jun 35	Flood	1.04	2.7 rise	—	3.3 rise	+1.5	4,600	4,600
	Ebb	1.28	2.8 fall	—	3.5 fall	-1.7	4,600	4,600

1 Positive values (flood): *negative values (ebb).*
2 MCCP data.
3 Historic data, USAED, Philadelphia, files.

Inlet, as well as wind effects that can cause flow exchange between the Manasquan River estuary and Barnegat Bay via the Point Pleasant Canal or between the Manasquan estuary and the ocean. However, both the 1982 and 1983 tidal prism measurements were obtained during fair weather conditions with low wind speeds and no rainfall. Thus, the tidal prism measurements primarily reflect typical spring tidal forcing of inlet flow.

A comparison of ebb- and flood-tidal prisms in 1982 and 1983 shows that the flood prism was larger than the ebb prism in both cases, with the 1983 flood prism larger than the 1982 flood prism. The principal factor accounting for the differences between the observed 1982 and 1983 flood-tidal prisms is the range of the tide occurring at the time. There is a direct but nonlinear relationship between the tide range and the corresponding tidal prism. A comparison of the tide ranges observed at the inlet throat (on the temporary tide staff at Sta 18+00) and at the USCG station tide gage location reflects the gradual attenuation in tidal range with increasing distance from the ocean. This response is typical of shallow estuaries connected to the sea through "narrow" tidal inlets such as Manasquan Inlet.

Maximum flow velocities averaged over the inlet cross section were higher in the 1983 measurements than in 1982. This effect is in part because of the larger flood-tidal prism in 1983 as compared with 1982, but also because of the relatively smaller cross-sectional area at Sta 18+00 in 1983 compared to 1982. However, the minimum (or controlling) cross-sectional areas were approximately equal in 1982 and 1983. The controlling cross section in Manasquan Inlet typically occurs near the seaward end of the channel, presumably because of the influx of littoral sediments passing around the jetty heads.

The tidal prism values observed in 1931 are about one-third of the 1982 and 1983 values. This is principally due to the relatively smaller channel geometry existing in 1931, and to the fact that the 1931 flow measurements were made during neap tide conditions. The 1935 tidal prism measurements were also made during neap tide conditions, but the larger tidal prisms compared with 1931 primarily reflect the increase in channel cross section due to dredging to the authorized channel dimensions. Although the controlling cross-sectional area in 1935 was not significantly smaller than in 1982 and 1983, the channel geometry was relatively uniform over the length of the inlet in 1935. In contrast, the channel geometry during the 1980s has been less uniform along the inlet axis, with most of the channel deeper than in 1935, but with localized shoals near the ocean entrance.

The relationship between the tidal hydraulics and channel geometry of a large number of inlets in the United States was evaluated by Jarrett (1976). A range of hydraulic and geometric parameters was compiled for each inlet. The inlets were then grouped according to location (e.g., Atlantic, Pacific, or gulf coasts) and presence or absence of jetties. A regression analysis was performed correlating the controlling inlet cross-sectional area

below MSL with the spring tidal prism. For the category "Atlantic Coast Inlets with Two Jetties," the regression equation for P (the tidal prism in units of cubic feet), versus A (the controlling cross-sectional area below MSL in units of square feet) was found to be:

$$A = 5.77 \times 10^{-5} P^{0.95} \tag{1}$$

A plot of this regression equation and the 95-percent confidence limits is shown on Figure 34. The 29 data points used by Jarrett (1976) in the regression analysis are indicated on the plot. The largest tidal prisms measured at Manasquan Inlet in each of the four data sets from Table 18 (i.e., 1931, 1935, 1982, and 1983) are also shown on Figure 34 at the appropriate value of controlling cross-sectional area. The 1931 and 1935 values plot relatively closer to the regression line than do the 1982 and 1983 values, despite the fact that the 1930s measurements were made during neap tide conditions. The 1982 and 1983 values plot above the regression line but within the 95-percent confidence limits. Presumably, if the 1931 and 1935 measurements had been made under spring tide conditions, the points on Figure 34 would plot well above the regression line, similar to the 1982 and 1983 points. However, both points would plot closer to the regression line if a larger controlling area was used. This probably reflects the fact that the controlling cross-sectional area at an isolated shoal feature is not as effective a "throttle" on the inlet's tidal prism as is a controlling section that occurs along a greater fraction of the inlet length.

Both sets of measurements demonstrated that despite the natural variability of the tidal prism under spring tide conditions. there is a stable range of values for the tidal prism. These measurements also demonstrated that the observed tidal prism-inlet area ratios at Manasquan Inlet agree well with the empirical equation for Atlantic coast inlets with two jetties (Jarrett 1976). These findings agree well with the long-term experience in the operation and maintenance of the Manasquan Inlet Navigation Project. Relatively little maintenance dredging has been required historically in the jettied channel area because the channel dimensions are in approximate equilibrium with the controlling tidal hydraulic and sedimentary regimes.

A survey of the underwater portions of the south jetty using the Philadelphia District's side-scan sonar (SSS) was attempted on 29-30 September 1982. This experiment was aimed at determining whether the SSS could resolve the complicated geometry of the submerged dolosse that were not observable by other means and, if so, whether the dolos positions could be monitored over a period of time. The effort indicated that SSS probably could not distinguish between the dolosse and stone armor, although two conditions contributed to the degradation of the imagery. First, sea conditions were rough, making conditions in the inlet marginal for SSS survey work. Second, the SSS was rigidly affixed to the hull of the boat in an attempt to prevent it from being damaged on the dolosse or rocks. This placement caused all the wave action to be transmitted to the transducer degrading the imagery.

A second attempt was made on 20 July 1984 to survey the entire jetty system using CERC's Klein 500kHz SSS, a unit that produced the best SSS imagery detail available at the time. There was a significant improvement in image quality (Figure 35), but heavy boat traffic and 2-to 3-ft swell reduced the effectiveness of the SSS. Imagery of both jetties was produced, but the best imagery was of the outside of the north jetty, since the waves were generally from the southeastern quadrant and the area was out of the influence of most boat traffic. Records of the north jetty head and the insides of both jetties were marginal, and the record of the south jetty head was unreadable. The problem was air entrained in the water column by breaking waves and boat propellers.

The structure appeared in good shape in the SSS record, even though it had experienced nearly the design storm only 4 months earlier. Several "holes," or areas where dolosse appeared to be missing, could be seen in the record. Rock displaced from the structure slope and, possibly, its core could be seen in the record. There was also some debris, possibly from a fishing boat that had been sunk in the inlet in a storm, noted on the record.

Some specific comments were:

a. An indentation was noted at the toe of the north jetty head at approximately the 65-deg radial (compass direction). No dolosse were visible on the bottom near the jetty head, so it is likely that the indentation was the result of poor placement.

b. Two holes were identified in the dolos cover on the north side of the north jetty. Several individual displaced rocks were identified on the bottom, along with one or two displaced rock/dolos fragments. This observation highlights one of the advantages of SSS as a tool. If there were concerns about the integrity of the structure, divers could be used to investigate those areas identified by the SSS as possible problem areas, thus reducing the time divers needed to be in the water. Diving surveys are costly and dangerous; SSS can be used to keep them to a minimum.

c. Two terraces, areas where the slope flattens, were identified near the waterline on the inside of the north jetty. These terraces started approximately 160 ft shoreward of the end of the dolos section.

d. Large stones at the toe of the slope were observed inside the south jetty. Sand waves, both ripples and waves with amplitudes of several feet, could be recognized. Finally, the rock- and dolos-armored slopes were easily distinguished in the record.

Inlet hydrographic surveys were obtained as a part of the monitoring effort. These, together with historic surveys, were used to evaluate changes in the inlet that resulted from jetty rehabilitation.

The Federal project at Manasquan Inlet was constructed to the current authorized dimensions in 1961. Sand originating from the local littoral environment subsequently shoaled within the inlet and was dredged by Government hopper dredges on an annual to semiannual basis until 1978. The annual rate of dredging was approximately 35,000 cu yd. These shoals typically formed within the first 1,300 ft of the inlet mouth, with the predominant shoaling occurring in the north half-channel. Controlling depths between dredge intervals were generally -10 ft (MLW). The interior of the inlet between Sta 13+00 and 21+36 exhibited less tendency to shoal and was instead relatively stable at depths between -15 ft and -19 ft (MLW). Dredging has not been performed in the inlet since July 1978, even though the channel shoaled approximately 50,000 cu yd by September 1983. Dredging was not initiated during this period because of commencement of work to rehabilitate the jetties.

Prior to the jetty reconstruction project, the south jetty structure had become a less effective sand barrier. The inner sheet-pile core had deteriorated to the extent that holes and severe corrosion were evident. The armor layer and core stone had been sufficiently displaced in some areas to create large voids enabling sand and water to easily pass through the structure. The outer end of the south jetty had also been displaced, perhaps allowing sand to migrate more easily around the seaward end of the south jetty and into the inlet channel. The north jetty had been reconstructed in 1968 and was still structurally sound when the dolos reconstruction was initiated. Even though the inner sheet-pile core had partially deteriorated, there was a large quantity of small core stone which would have served as an effective sand barrier. Therefore, the rapid shoaling observed in the inlet channel adjacent to the north jetty between Sta 2+00 and 13+00 was probably not due to sand migrating through the north jetty, but more likely due to tidal current flow patterns causing deposition along this zone.

The inlet maintained a more stable cross-sectional area during the monitoring program than it did during the period prior to jetty reconstruction, even though distinct zones of both shoaling and scouring were evident within the inlet channel. Between August 1980 and April 1984, shoaling occurred between Sta 2+00 and 8+00 near the mouth of the inlet; however, the channel then scoured shoreward of this point to Sta 13+00. The interior portion of the inlet between Sta 13+00 and 21+36 remained relatively stable at depths between -15 and -19 ft (MLW), alternating between periods of minor shoaling and minor scouring. The total volume change within the inlet between these two dates was approximately 5,000 cu yd.

A series of surveys taken during the reconstruction of the north and south jetties and during the monitoring period after completion of the project identified a temporal pattern of shoaling and scouring. During construction of the project, the inlet shoaled significantly from the after-dredging conditions of July 1978. A survey conducted in October 1982, soon after the completion of the north jetty, revealed that large shoals had formed in the inlet between Sta 2+00 and 11+00. Controlling depths were

-9 to -11 ft (MLW) with a minimum depth of -8 ft (MLW). However, subsequent surveys revealed that these shoals then progressively scoured, resulting in controlling depths of -13 to -14 ft (MLW) with a minimum depth of -12 ft (MLW) in April 1984.

While the inlet alternated between scour and slight shoaling from October 1982 through April 1984, the 5-month period from April to September 1984 generated approximately 20,000 cu yd of shoaling. Controlling depths reached -10 ft (MLW) with a minimum of -8.5 ft (MLW). Much of this material was again naturally flushed from the channel by tidal flow such that by January 1986, approximately 10,000 cu yd had scoured, primarily between Sta 1+00 and 8+00. Perhaps the increased flow velocities associated with the storm surge from Hurricane Gloria in September 1985 may have caused the observed scour between September 1984 and January 1986. This cannot be confirmed, however, because no hydrographic surveys were taken just prior to and after this storm event.

The inlet has continued to maintain a semistable cross-sectional area since the end of the MCCP program. Between January 1986 and April 1987, both scouring and shoaling were evident within the channel without any significant change in inlet morphology. Spot shoals identified in the 1986 surveys either maintained their general form and extent or accumulated material slightly. Other zones scoured and resulted in a total volume loss for the inlet channel of approximately 2,000 cu yd. Controlling depths as of April 1987 were -11 ft (MLW) with a minimum depth of -10.5 ft (MLW).

The most obvious and persistent morphological feature of the channel bathymetry has been the development of a quasi-stable undulating bottom. A sand wave pattern has developed from a series of sand ridges and valleys within the inlet channel. Between Sta 1+00 and 13+00, two distinct shoals had begun to develop by October 1982. These shoals then went through alternating periods of accumulation and scour and by April 1987 had developed into two prominent sand ridges. Depths across these ridges were about -12 to -14 ft (MLW) with minimum depths of -10 ft (MLW). Depths on either side of these shoals were between -16 ft and -19 ft (MLW).

Within the interior portion of the inlet, two smaller spot shoals, also in the form of sand waves, began to develop by September 1984 on the south side of the inlet at Sta 15+00 and 18+00, respectively. These shoals migrated toward the interior of the inlet and were opposite Sta 17+00 and 20+00 by April 1987. The north half of the channel between Sta 13+00 and 21+36 has remained relatively stable with depths typically between -16 and -19 ft (MLW).

As mentioned, the two prominent shoals located near the mouth of the inlet in April 1987 had started to develop several years earlier, or soon after the last inlet dredging in July 1978. Therefore, it is difficult to resolve whether the present shoaling pattern is unique to the new hydraulic

environment as modified by the jetty reconstruction, or simply a more stable configuration of the shoals that were already forming during the construction of these structures between 1978 and 1982.

Sand tightening of the jetties has produced a significant improvement in maintenance of the channel. Prior to the rehabilitation, the channel had to be dredged every 1-1/2 years on average. Since the project, no dredging has been required in Manasquan Inlet. Figures 36-39 show conditions in the inlet at various times from prior to project approval to after completion of construction. Figure 36 shows bathymetric conditions in the inlet in April 1977. Figures 37 and 38 show conditions just after and 6 months after the northeaster of March 1984. Figure 39 shows conditions in January 1986.

Beach profiles were obtained at the 17 profile line locations shown on Figure 9 on nine discrete occasions between September 1982 and April 1984, as listed in Table 3. On four of the nine survey dates, combined onshore and offshore profiles were obtained, whereas on the other five dates, only onshore data were obtained. The beach profile data provide the basis for evaluating the short-term variability of the beach configuration north and south of the inlet. The data also provide a means to examine the longer term evolution of the study area shoreline through a comparison of the present shoreline alignment and configuration with those that are documented in historic records.

The time intervals between the onshore surveys ranged from a minimum of 6 days, which is the time between before- and after-storm surveys for the March 1984 northeaster, to a maximum of 128 days between the November 1982 and March 1983 surveys. The mean time interval between survey dates was 69 days (0.19 year) over the course of the 1.5 years of repetitive beach profiling. In this period, two complete winter seasons were included (1982-83 and 1983-84 winters) while only a single full summer season (1983) was documented. The first eight survey dates are viewed as characterizing the range of seasonal conditions typical for this region of the New Jersey coast, whereas the final two survey dates characterize the effects of a single, large storm event with an estimated 25-year return period.

The survey data were input and analyzed using the CE's Interactive Survey Reduction Program (ISRP). This software package allows rapid calculation of the two most useful parameters describing beach profile changes as a function of time: shoreline location change, adopted in this effort as the locations of the 0-ft NGVD contour, and onshore profile volume change, adopted as the volume change in cubic yards per lineal foot of beach above the NGVD datum at each profile line.

The short-term variability of the beach profiles is illustrated in Figure 40, displaying the range of shoreline locations at each line, and in Figure 41, displaying the profile volume envelope at each line. Each of these figures presents two conditions: the hatched bars represent profile variability

over the entire period of record (1.5 years) of the monitoring program, including effects of the March 1984 storm. The open bars represent the profile variability for the period of record except that the final, poststorm survey data set was not included. The potential effects of a single event are demonstrated most vividly on the profile volume envelope plot in Figure 41. It can be seen that the profile variability over time can be dominated by the changes from a single day as compared with changes that occur over periods of a year or more under "typical" conditions. The plot of shoreline excursion shows much less sensitivity to the March 1984 storm effects, primarily because of the displacement of material from higher up on the profile down to the intertidal and near offshore zone, with less shoreline retreat resulting than occurred with the sediment volume above NGVD.

Figures 40 and 41 also display significant differences between the updrift, unstructured Point Pleasant and downdrift groin-segmented Manasquan shorelines. It can be seen that the range of variability, particularly the profile envelope volume, decreases in a relatively regular manner at Point Pleasant with increasing distance south from the inlet. The greatest variability at Point Pleasant is in the south jetty fillet area (profiles PP-1 and PP-2), with the minimum variability at the southernmost profile locations (PP-7 and PP-8). In contrast, there is no systematic trend in profile variability evident at Manasquan with distance from the inlet. The most reasonable explanation for the difference in behavior of the two beaches lies in the influence of the groin field present at Manasquan, consisting of six stone and eight wooden groins. These structures reduce at least the longshore, if not the cross-shore, transport of sediment under the range of conditions experienced during the monitoring period.

The beach profile data were also used to compare the "average" present conditions of the Point Pleasant and Manasquan shorelines with those documented on historic surveys. Within the limits of the area monitored in this study, historic cross sections of the beach and offshore were available for 1931 (prejetty construction), 1935 (postjetty completion), 1953, 1963, 1965, and 1978. Although not every survey data location includes as many profiles as were monitored during 1982-84, there are sufficient data, particularly from 1931 and 1935, to reveal the principal features of the updrift and downdrift beach evolution in response to the construction of the jetties. The historic survey data also permit an evaluation of almost 50 additional years of long-term shoreline evolution (1935-1984) since the jetties have been in place. Table 15 summarized the shoreline location changes at each profile line for the 1931-1935 period, showing the immediate effects of jetty construction, compared with the total change measured from 1931 (prejetties) to 1983, which is the survey date that best typifies the average conditions during the 1982-84 monitoring period.

The data in this table display the immediate short-term effects of the jetty construction at Manaquan Inlet, which include rapid shoreline accretion on the immediate updrift (south) side of the inlet and the shoreline loss on

Table 15
Manasquan Inlet Vicinity: Shoreline Location Evolution, 1931-1935-1983, Shoreline Position Change (0-ft NGVD Contour)

	Profile Id	1931-1935	1935-1983	1981-1983	1931-1983 Change 1982-84, 0-ft NGVD Range
North (Downdrift)	M-9	0	-20	-20	0.2
	M-8	0	0	0	N/A
	M-7	-30	+40	+10	0.2
	M-6	-30	0	-30	0.4
	M-5	-70	+40	-30	0.3
	M-4	-110	0	-110	1.6
	M-3	-120	0	-120	2.0
	M-2	-180	+40	-140	2.2
	M-1	-150	+40	-110	1.6
			Manasquan Inlet		
South (Updrift)	PP-1	+200	+120	+320	1.9
	PP-2	+200	+170	+370	2.9
	PP-3	+150	+200	+350	3.7
	PP-4	+110	+210	+320	4.6
	PP-5	+70	+200	+270	3.9
	PP-6	+30	+210	+240	4.8
	PP-7	+10	+170	+180	3.6
	PP-8	-20	+160	+140	4.7

the downdrift (north) side. On both sides of the inlet, the immediate effects are most pronounced at profile line locations nearest the inlet, with little or no effect detectable at distances from the jetties. The 1935-1983 changes reflect the longer term evolution of the shorelines since the jetties were completed. The final two columns show the net change since 1931 and the ratio of this value to the short-term variability as documented by the 1982-84 profile data. The larger values represent locations at which there has been a statistically significant long-term shoreline trend as compared with the potential short-term variability. This effect is most pronounced at the extreme updrift (south) end of the study area in southern Point Pleasant, while in northern Point Pleasant and in Manasquan, lower values of long-term change are evident.

Aerial photography obtained during the monitoring effort provided a source of qualitative information on the project and its effects on the adjacent shorelines. As expected, there was little shoreline response to the project, since it involved only the rehabilitation of existing, albeit deteriorated, structures.

During the same flights, lower altitude photography was taken for the photogrammetric analysis of armor unit stability. Determining the accuracy of the photogrammetric techniques involved analysis of both the photogrammetric and leveling measurements which were compiled independently. Prior to the 7-8 September 1983 leveling and 15 September 1983 photography, the comparison of photogrammetric and standard leveling data suggested that the accuracy of the photogrammetrically derived elevations was on the order of +/-0.3 ft. However, there were two factors identified that could have been contributing to the differences between leveling and photogrammetric elevation data. The first factor was that the dates of leveling differed by as much as 2 or 3 months from the closest photography. It was possible that dolos movement had occurred during those periods, contributing to apparent differences between photogrammetric and leveling measurements of the same point. For example, the February 1983 storm occurred in the 6-week interval between the January 1983 photography and the March 1983 leveling. The second factor was that prior to September 1983, there were no visual targets on the dolosse to ensure that the survey crew and the photogrammetrist were observing exactly the same point when measuring an elevation. Features such as "center of face of vertical fluke" were the nominal targets used by surveyors and photogrammetrist for identifying locations for spot elevations. If the surface was inclined relative to horizontal, as are almost all dolos surfaces on a jetty, small differences in horizontal location could contribute comparable differences in measured elevation.

The best data set for determining the accuracy of photogrammetric elevations was obtained in the storm-free period of 7-15 September 1983, when both leveling measurements and aerial photography were performed. For these observations, 1-ft black crosses were painted on 111 dolosse distributed over the two jetties, assuring that both the field crew and the photogrammetrist would determine elevations at the same points on the units. Elevations were determined to the nearest 0.01 ft for both methods. Comparison of the elevation data from the two methods demonstrated that 84 percent of the photogrammetric values were within +/-0.1 ft of the elevations determined by leveling and 98 percent were within +/-0.2 ft. The largest discrepancy between the two methods at any point was 0.27 ft. These findings strongly suggested that earlier uncertainties regarding accuracy and resolution of the photogrammetric elevations were due to the time interval between measurements and the lack of point targets on the dolosse. The findings also showed that photogrammetry was capable of accurately resolving a scale of movement of individual armor units that would permit a detailed evaluation of dolos stability.

Leveling data were essential in verifying the accuracy of the photogrammetric elevations. However, the leveling data do not provide any information on horizontal displacement, whereas both elevation and planimetric information are provided by photogrammetry. Nevertheless, the leveling data summarized in Figure 42 suggest a relationship between dolos movement and storm exposure. Note that level measurements on the south jetty were obtained from April 1982 to September 1983, during which time two

northeasters occurred (October 1982 and February 1983). The north jetty level measurements were obtained between March and September 1983, a relatively storm-free period. The data show that the south jetty dolosse experienced more frequent and greater downward vertical displacements than did the north jetty dolosse.

South jetty maps S1, S2, and S3 and north jetty maps N1 and N2 were prepared from aerial photography obtained through September 1983. As previously discussed, these photogrammetric maps did not achieve as high a degree of accuracy in measuring dolos movement as did later maps. However, analysis of the photogrammetric displacement data through September 1983 did show that 65 percent of the 250 observed points were within 0.3 ft and 91 percent were within 1.0 ft of their initial elevations. The maximum vertical change detected was a drop of 4.2 ft on a dolosse at the head of the south jetty. Ninety percent of the vertical displacements that exceeded 1.0 ft occurred on dolosse at the heads of the two structures. The largest horizontal displacement detected was nearly 6.0 ft on a dolosse on the channel side of the south jetty. The next largest horizontal displacement was only 3.5 ft, occurring on the head of the south jetty. The mean horizontal movement of all monitored dolosse through September 1983 was about 1.0 ft. The movements were predominantly rotation around the vertical axis (yaw) and displacement in a downslope direction relative to the structures.

All photogrammetric measurements on maps for the period from September 1983 through May 1984 used targets established in September 1983 and are therefore assumed to be of comparable accuracy. Note from Table 4 that the period from 15 September 1983 to 27 March 1984 was relatively storm free, whereas the interval from 27 March to 9 May 1984 was not. Measurements of vertical and horizontal displacements over these two intervals reinforced the earlier findings that dolos movements were predominantly related to storm effects.

In the 6-month period from 15 September 1983 to 27 March 1984, the mean vertical displacement for all points monitored on the two jetties was 0.15 ft, and only 10 percent of the monitored dolosse experienced detectable horizontal displacements, the largest of which was about 1.0 ft.

Between 28 and 30 March 1984, an intense coastal storm affected the mid-Atlantic states. During the 29th, there was a maximum of about 6.0 ft of storm surge above the predicted tide levels. The maximum still-water elevation recorded at the long-term tide gage at Atlantic City, about 50 miles to the south, was only 0.1 ft below that attained during the March 1962 "storm of record." The maximum stage recorded at Manasquan was 0.2 ft below that at Atlantic City on 29 March 1984. However, coastal damage was less in March 1984 because the highest waves coincided with only one high tide, in contrast to the "five high tide" duration of the March 1962 storm. The wave gage at Manasquan, located about 1 mile northeast of the inlet in about 50 ft of water, recorded a maximum 20-min

significant wave height of about 22 ft with a corresponding peak period of about 11.5 sec. The peak of the wave record coincided with the maximum of the ocean stage, and thus exposed the jetties to what is believed to be the equivalent of the design storm The significant wave height at the gage exceeded 20 ft for 5 hr and exceeded 10 ft for 30 hr. Note that the 27 March 1984, photography was obtained only 24 hr before the storm began affecting Manasquan.

The mean vertical displacement of all monitored dolosse because of the March 1984 storm was -0.46 ft. Figure 43 summarizes the elevation change data from this storm. Approximately 3 percent of the dolosse moved in excess of 1.0 ft vertically, with a maximum value indicating a 2.03-ft drop.

The largest horizontal displacement caused by the March 1984 storm was 7.0 ft at the head of the south jetty. There were three other dolosse that moved about 5 ft horizontally. Altogether, only 9 percent of the monitored dolosse moved in excess of 2 ft horizontally, with 31 percent moving up to 2 ft. About 60 percent of the dolosse experienced no detectable horizontal displacement.

As a result of the March 1984 storm, three dolosse broke on the north jetty, all within a zone about 35 ft wide at the head of the structure. Two of the breaks resulted in the loss of some concrete from the shank portions of the dolosse, but the presence of the epoxy-coated reinforcing steel kept the dolosse substantially intact. One of these dolosse was significantly damaged, with considerable loss of concrete. After the storm, all the reinforcing steel was exposed in the break. During an inspection in July 1989, the dolos was found to be completely separated. The reinforcing steel had been twisted until it had broken (Figure 44). The third north jetty dolos suffered a hairline crack through one fluke. One south jetty dolos, located near the head on the channel side of the structure, broke at the junction of the shank and fluke. This dolos is also essentially intact because of the reinforcing steel.

Prior to the March 1984 storm, one other dolos at the head of the north jetty had broken. Therefore, a total of 5 of the 1,326 dolosse (only 0.4 percent) used in the 1979-1982 rehabilitation have broken despite exposure to the design storm event. It should be noted that four of the five dolosse failed through the shank. No additional visible dolosse had broken as late as July 1989. Four dolosse, broken during south jetty rehabilitation drop tests, had the breaks epoxied and were placed in less exposed locations on the jetties. None of these dolosse have broken. It is interesting to note that of the five dolosse that have broken, only one has experienced a net horizontal displacement in excess of 2 ft from its initial location. Other dolosse have moved greater distances, up to 7 ft between successive photography, yet have not broken. This finding seems to support the contention that movement alone is not responsible for armor unit breakage. Impact appears

much more important than movement in dolos breakage. An armor unit can experience significant impacts even with only small movements.

Based on several years of monitoring at Manasquan Inlet, it is evident that steel-reinforced dolosse weighing 16 tons can exhibit a degree of mobility on the jetty face in response to storm conditions and not incur significant damage because of breakage of individual units.

One concern about the use of reinforced dolosse is that the reinforcing steel will eventually be affected by salt water seeping into the unit through cracks of any size. The resultant corrosion could cause the reinforcing rod to swell and structurally compromise the unit from within. Even though the last dolosse were placed at Manasquan in 1982, there is little evidence of rusting reinforcing steel. A number of units (although well less than half those visible) have small rust stains, principally on the flukes. Surprisingly, the cracked dolosse show little evidence of rust even near the cracks (Figure 45). Although it may still be too early to determine if the dolosse at Manasquan are suffering from rusting reinforcing steel, they have performed admirably to date. Since rehabilitation, the structures have required no maintenance. This is a significant change from the effort required on the structures historically. From the time the jetties were constructed until the rehabilitation, the jetties required maintenance every 3.5 years on average. A total of $5,120,100 (1978 dollars) was spent on repairs to the jetties, or an average of about $500,000/repair. During the last 10 years before the rehabilitation, the jetties were not repaired, being allowed to deteriorate until the rehabilitation was begun. Assuming that another $2,500,000 would have been spent over the years since 1968, the year of the last repair, and the present, the $5.7 million spent on the rehabilitation seems to be a cost-effective alternative to continued repair. In the Design Memorandum for the rehabilitation, it was estimated that $20,000 would be spent per year on maintenance of the structure. In fact, no maintenance has been required since the rehabilitation, and none appears to be needed, at least in the near future. There are a few qualitative observations that might be made about the motions of the dolosse in the period up to the March 1984 storm and then as a result of the storm. Prior to the storm, the head sections were the most active, as might be expected, with primary motion being downslope, i.e. a loss of elevation and movement away from the jetty center line. Along the trunks of the structures, a larger percentage of the motion was into the jetty, toward the center line, and along the jetty, particularly shoreward.

During the storm, the head of the south jetty and the south side of the jetty showed mostly movement downslope. Dolosse on the channel side of the head of the south jetty, as well as on the trunk, moved along the jetty. On the north side, movement at the head was more mixed than on the south jetty, but motion downslope predominated. As on the south jetty, dolosse on the trunk moved mostly along the structure.

As has been already discussed, most dolos movements were small, so much of the movement measured was of little consequence when considering structural stability. By far, the largest movements were down the slope of the jetty, but even those were few enough not to jeopardize the structure.

Larson and Kraus (1989) describe the data set for the March 1984 storm as "perhaps the most complete data set on storm-induced erosion obtained to date in the United States." The data set includes continuous wave data, obtained hourly during the storm, a complete water level record, wading surveys just before the storm, complete profiles within days of the storm, and aerial photography prior to and after the storm. Much of the information collected during the storm has been presented previously; however, Figure 46 shows a time-history of the significant wave heights during the storm. The highest significant wave height was over 20 ft for 5 hr during the storm. Information on the volume of beach material lost and horizontal retreat and vertical loss for both Manasquan and Point Pleasant is contained in Table 16. Because of their potential interest, the pre- and poststorm profiles are included for all profile lines for Point Pleasant (Figures 47-54) and Manasquan (Figures 55-63). As can be seen from Table 16 and Figures 47-63, the beach at Manasquan was less affected than that at Point Pleasant, most likely as a result of the groins at Manasquan. Once again, it must be noted that by the time of the poststorm profiles, the beach had already experienced several days of recovery.

The storm approached the conditions of the design storm for the dolos rehabilitation, so it provides an excellent test of the structure and its dolos armor. The response of the structure was excellent. There was some movement of the dolos armor, as would be expected, but the integrity of the structure was maintained. No repairs were necessary to the structure nor were any needed at the time of the last inspection in July 1989. The northeaster of March 1984 provided an unusual opportunity to monitor a structure during its design event.

Table 16
Profile Changes Due to 28-30 March 1984 Storm

Profile Number	Volume Lost[1] cu yd/lin ft of Beach	Max Horizontal Retreat, ft	Max Vertical Loss, ft	Width of Erosion Zone, ft
Manasquan Inlet				
M-1	12.1	60	3.9	120
M-2	13.1	65	3.1	170
M-3	17.8	60	5.4	120
M-4	32.4	75	6.4	190
M-5	20.6	60	5.0	170
M-6	14.1	60	3.9	140
M-7	8.2	55	3.9	80
M-8	15.3	50	4.5	140
M-9	14.0	70	3.7	160
Average	16.4	62	4.4	143
Point Pleasant				
PP-1	6.2	50	4.0	75
PP-2	26.0	150	4.4	250
PP-3	29.9	135	5.0	270
PP-4	25.7	125	6.5	200
PP-5	26.1	125	6.1	210
PP-6	17.0	100	7.2	120
PP-7	25.9	125	8.3	160
PP-8	21.9	105	8.0	150
Average	22.3	114	6.2	179

1 Above approximately -2 ft NGVD.

3 Related Studies

Two studies similar to that at Manasquan Inlet have been recently completed or are nearing completion. Another dolos study performed under the MCCP Program was that at Cleveland, OH (Figure 64), where the easternmost 4,000 ft of the breakwater was rehabilitated with 2-ton, unreinforced dolosse. At Cleveland, there was also an attempt to quantify dolos movement and breakage. The report for that study is in preparation, but some interesting results have been noted. The percentage of dolosse broken at Cleveland was far greater than at Manasquan. In fact, the dolos breakage was a matter of concern at Cleveland. It took several years for the breakage to slow (Figure 65), indicating that the structure was finally attaining a degree of stability. In fact, the head of the structure had to be reconstructed several times after sustaining severe damage in every major storm during and after the monitoring period.

There were attempts to correlate dolos breakage and movement to waves, lake levels, and ice. Little data were acquired to relate damage to waves, unfortunately, because the gages had to be removed during the winter to protect them from ice. It was, of course, during the winter months that the worst storms were experienced. As can be seen from Figure 66, there is little correlation between variations in lake levels and damage to armor units, although one instance of head damage did occur during high lake levels and may have been caused by the concrete monolith on the east head. There does seem to be a correlation between breakage and ice. Figure 67 shows this rough correlation. While the data are still being evaluated, it appears that ice contributes to the dolos breakage at Cleveland, but waves are likely to be the principal cause.

Finally, there were repeated surveys of targeted dolosse on the breakwater. Figure 68 shows a portion of the results of those surveys. Some general observations on dolos movement have been made. Dolos movement was fairly uniform over the entire structure, with the general movement downslope. Dolosse on the crest that were not keyed into the structure were often thrown onto the jetty crest or even over the structure. Dolosse at the waterline generally moved more than those on the rest of the structure. Unfortunately, there was little reduction in motion of the units with time.

The most extensive monitoring of dolos armor units has been undertaken at Crescent City, CA (Figure 69), by the Crescent City Prototype Dolosse Study (CCPDS). Data have been collected on waves, winds, dolos breakage, and dolos movement (through photogrammetry). However, the most impressive data set collected has been from strain gages and accelerometers mounted inside a number of the 42-ton dolosse (Figure 70). This is a landmark data set that will contribute a great deal to the understanding of dolos performance and structural design.

Analysis of the Crescent City dolos load data revealed that during the CCPDS no impacts between dolosse were recorded, but large amounts of data were collected for static loads and pulsating wave loads (Howell et al., in preparation). Further analysis revealed that the pulsating wave-induced stresses in the dolos could be approximated by a Rayleigh-distribution. Crescent City dolos, maximum principal stresses can now be predicted as a function of significant wave height. The CCPDS culminated in a Crescent City dolos design procedure (Melby, in preparation) that can be used when and if future dolos repairs are needed at Crescent City. The prototype dolos and wave data were also used to validate an instrumental model dolos technology (Markle and Greer, in preparation) that can be used to determine wave-induced dolos loadings for Crescent City and other project sites as well as being used in applied research studies being conducted to develop generalized dolos structural design guidance.

With the end of the CCPDS Study, dolos monitoring was continued under the MCCP work unit entitled "Periodic Inspections." The scope of work for monitoring was reduced relative to the CCPDS, but dolos movement and static load data along with wave data from the Point St. George, NOAA Buoy will continue to be collected through FY92. Results of dolos movement and static load data analysis through FY90 show that dolos movement is diminishing, but static loads are continuing to increase and are nearing critical levels in some dolosse (WES 1991).

Markle and Davidson (1983) evaluated the effect of armor unit breakage on the stability of a two layer, randomly placed, dolos-armored breakwater trunk with a 1v:1.5h slope. Their conclusion was that there was a threshold of breakage below which breakage had no effect on the stability of the structure. That threshold was breakage of less than 15-percent uniform breakage in the top layer, 15-percent uniform breakage in the bottom layer, 7.5-percent uniform breakage in both layers, or clusters of five broken dolosse. Manasquan is certainly below this threshold, and the structure has remained stable, so it appears that these jetties support the conclusions of Markle and Davidson, even though the structure types are different.

4 Conclusions and Recommendations

Conclusions

The rehabilitation of the jetties at Manasquan Inlet, NJ, was monitored under the MCCP Program. Even though the structures have experienced a near-design storm, they have continued to perform successfully and have not required even the low level of maintenance anticipated by the designers. This overall excellent performance of the jetties and, in particular, the low percentage of broken dolosse during the March 1984 storm serve to verify the design and construction procedures used in the rehabilitation.

Markle and Davidson (1983) concluded that there was a threshold of breakage of a dolos-armored structure beyond which the structure was likely to fail. The jetties at Manasquan are below this threshold and have remained stable even through a near-design storm.

Use of photogrammetric mapping of the jetties allowed a detailed evaluation of the motion of the armor units. This technique was found to be cost-effective and accurate, providing accuracy comparable with standard leveling techniques. Comparison of the results of this study with preliminary results from Crescent City, CA, seems to verify an hypothesis made by Kendall (1988) that dolos armor units on flatter slopes tend to be forced up the slope by forces associated with wave runup, while those on steeper slopes, such as at Manasquan, will be moved downslope by wave rundown.

It is apparent that the dolosse at Manasquan Inlet have benefitted from the use of steel reinforcement. Even those units that have cracked have been kept whole by their reinforcement. There are signs that the reinforcing steel may be rusting. This can be seen only on a relatively small number of units, so it too early to speculate on the fate of the dolosse. Reinforcing escalates the cost of casting dolosse, so the decision whether to reinforce the units is still one of cost/benefit, although EM 1110-2-2904 (US Army Corps of Engineers 1986) provided a rule of thumb for reinforcement. The work being done at Crescent City, CA, will provide some

insight into what size dolosse should be reinforced. At present, the largest dolosse are often designed for no impacts. However, the use of much of their unreinforced tensile strength is for supporting static loads. Smaller units will certainly move around and could benefit the most from reinforcement. The decision to reinforce dolosse armor units will continue to be based on engineering judgment until more information is acquired concerning the long-term effects of rust, the benefits associated with units maintaining their integrity even though cracked, and a better understanding of the relationship between impact load, static load, pulsating wave load, and dolos breakage.

Through the monitoring program, the value of sand-tightening the jetties was demonstrated. The jetties, particularly the south jetty, were quite porous, allowing considerable sand through the structure into the channel. There has been no maintenance dredging in Manasquan Inlet since the rehabilitation, another testimony to the design and the concept of sand-tight structures. The monitoring has shown that the sand-tight structures have had little apparent effect on the tidal prism.

Any monitoring program is at the whim of nature. Such an effort must have a finite life, during which it is hoped that there will be a significant test of the structures. At Manasquan, there was such a test. The rehabilitation survived a near-design storm in late March 1984. A particular success of the monitoring was the collection of an excellent storm data set, one of the most complete ever collected in the United States.

Based on its success at Cleveland, OH, side-scan sonar was used at Manasquan to evaluate the condition of the underwater portions of the structures. Although more was learned about the limitations of SSS than about its usefulness, the potential of SSS as a cost-effective inspection tool was reaffirmed.

One of the particular successes of the monitoring effort was the application of photogrammetric mapping to surveying the structural condition of coastal structures. While it has been applied to dolos armor in this study, it is equally applicable to structures with any type of natural or man-made armor. The accuracy of photogrammetry is more than adequate to evaluate armor unit movement. Periodic mapping of a coastal structure would permit detection of incipient or progressive failure along any visible portion of the structure before such a problem was readily detected by other means. This detection would allow for early assessment and possible correction of the problem.

Photogrammetry offers several advantages over conventional land surveying techniques. First, it is possible to map armor units at or near the waterline of the structure, units that would be inaccessible or too hazardous to reach on foot. Second, photogrammetry is flexible in that all the information needed to perform the mapping can be obtained almost instantaneously, permanently, and at fixed cost with one aerial photographic flight. The

mapping can then be performed at any time thereafter or not at all, depending on available resources, need for information, etc. In contrast, land survey methods capable of obtaining the location, orientation, and elevation data for mapping every visible armor unit are labor-intensive and would require more time and expense than photogrammetry. Had both base maps been prepared at the same time at Manasquan, the total cost of the initial, and most detailed, mapping of the jetties would have been about $6,000. For that amount, a map was produced of all visible dolosse with the positions of several points established on each of the 754 dolosse. The cost of leveling a total of 160 dolosse was estimated to be about $3,000. That cost is half that of the photogrammetry but produced elevations only on less than 21 percent of the visible dolosse. With the wider use of total stations, it is now possible to rapidly obtain position data using what has become standard surveying methods, but it is unlikely that improvements in survey techniques will reduce costs enough to challenge the cost effectiveness of photogrammetry.

A final advantage of photogrammetry is that the product is graphical. It is, therefore, more readily interpreted with respect to location and magnitude of armor unit displacements.

Despite the relatively short duration of monitoring, measurements have shown that the jetties have experienced a near-design storm. Photogrammetric measurements document that the dolosse do move on these jetties, especially in response to storm exposure. These measurements have quantitatively shown which dolosse have moved, how far, and in which direction. However, there is no indication that the range of dolosse displacements experienced to date has in any way compromised the effectiveness of the rehabilitation. The photogrammetric measurements have also shown that none of the monitored dolosse have experienced a displacement, either horizontal or vertical, in excess of about 65 percent of the unit dimension of 11 ft.

Recommendations

The monitoring effort at Manasquan Inlet has been quite successful. Data obtained have verified the excellent performance of the rehabilitated jetties, even in a near-design event; photogrammetry has been shown as a viable technique for monitoring the stability of coastal structures; and additional information has been gathered concerning design techniques used by coastal engineers.

Reinforcement of dolosse remains a matter of engineering judgment. Additional information is needed before guidelines can be developed for the use of reinforcing steel in dolosse. The USAED, Philadelphia, is encouraged to continue to evaluate the condition of the dolosse at Manasquan during site

visits. Emphasis should be placed on indications that the reinforcing steel is rusting, including rust stains and spalled concrete.

Periodically, on the order of every 5 years, the jetties should be photogrammetrically mapped. This mapping will provide additional useful information on the long-term stability of dolosse.

Sand-tightening the jetties at Manasquan Inlet has eliminated the need for maintenance dredging of the navigation channel. In situations where porous structures contribute to shoaling of a channel, the economics of rehabilitating the structures should be investigated.

Use of Jarrett's (1976) equation relating critical inlet cross-sectional area and tidal prism is appropriate for inlets that have exhibited historic stability.

Based on the studies that have been performed on sand transport in the Manasquan Inlet area, the use of WIS data, as applied by Gravens, Scheffner, and Hubertz (1989) seems to have the most potential for predicting sand transport with reasonable accuracy. LEO should be used for calculating sand transport with caution, because of the inherent inaccuracies involved in making the observations.

The data set from the March 1984 northeaster is one of the most complete data sets available in the United States. Researchers are encouraged to make use of these data and those collected at the CERC Field Research Facility during Hurricane Gloria in their work.

Side-scan sonar is an excellent tool for surveying the underwater portions of coastal structures. It is recommended for inspecting quality control during underwater placement of armor or for identifying problem areas after construction.

Photogrammetry has been shown to be a quick, cost-effective method for monitoring armor unit movement. Its expanded use could help all coastal CE offices to better monitor their structures.

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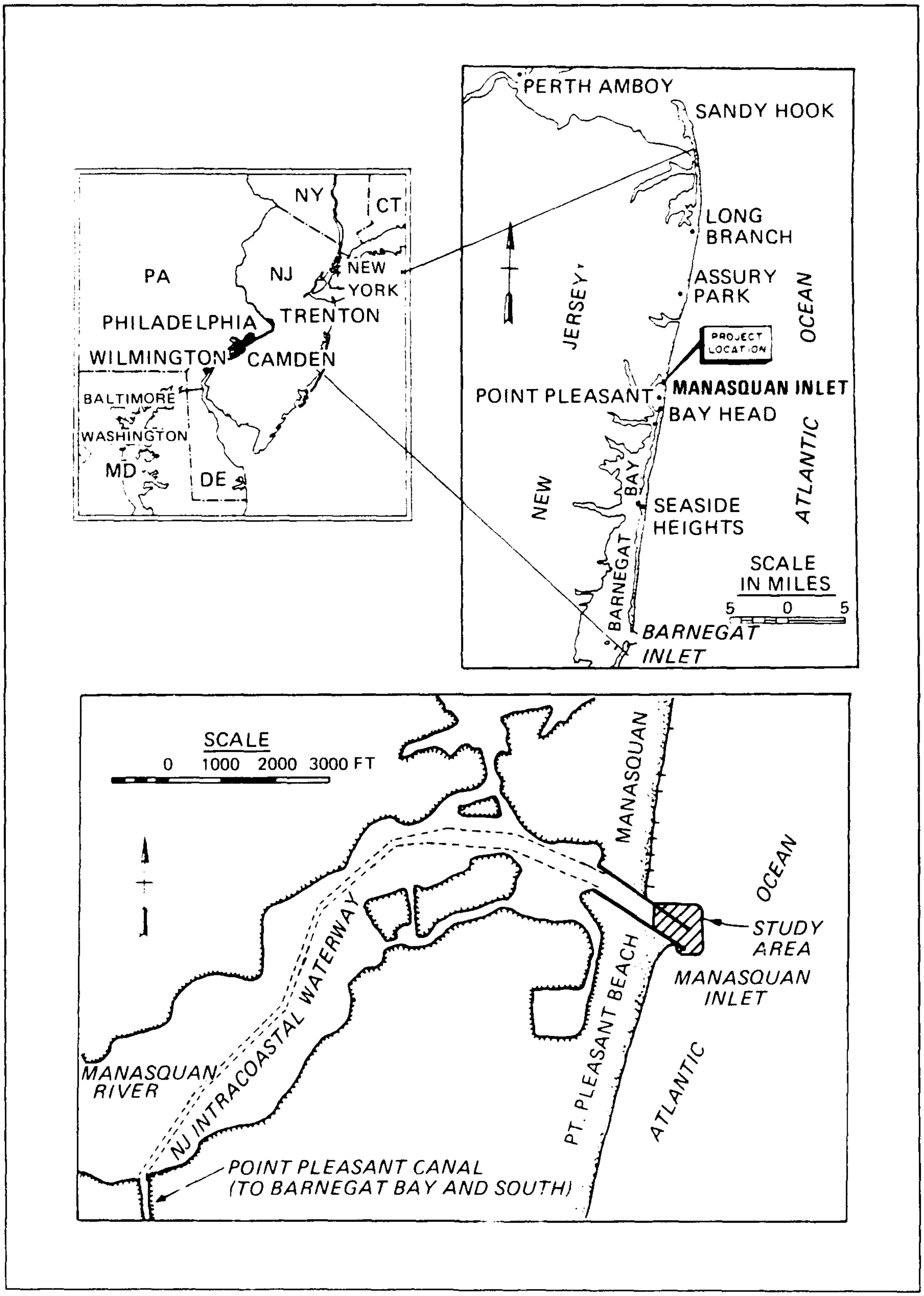


Figure 1. Location and vicinity map

Figure 2. Deteriorated jetties at Manasquan Inlet, 9 March 1962

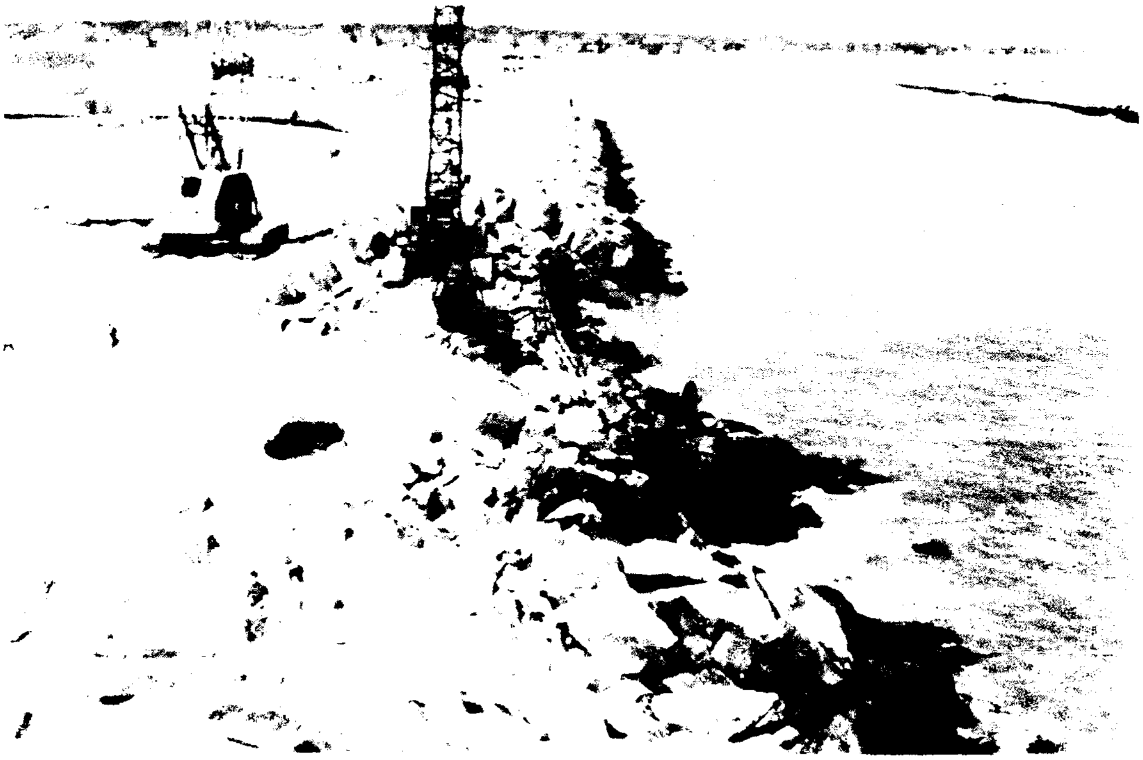


Figure 3. Reshaping of the jetty prior to dolos placement

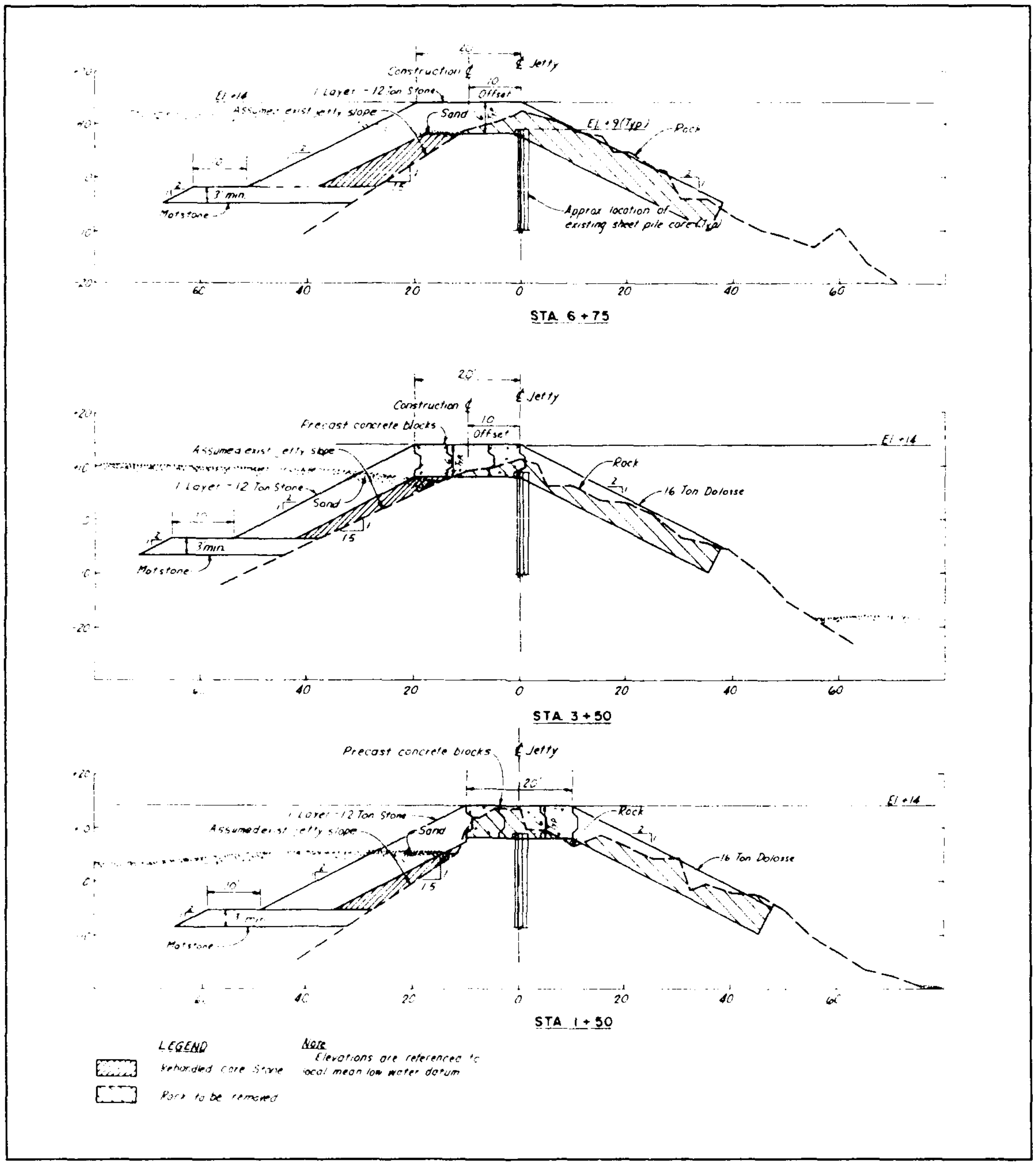


Figure 4. Construction drawings of south jetty cross sections

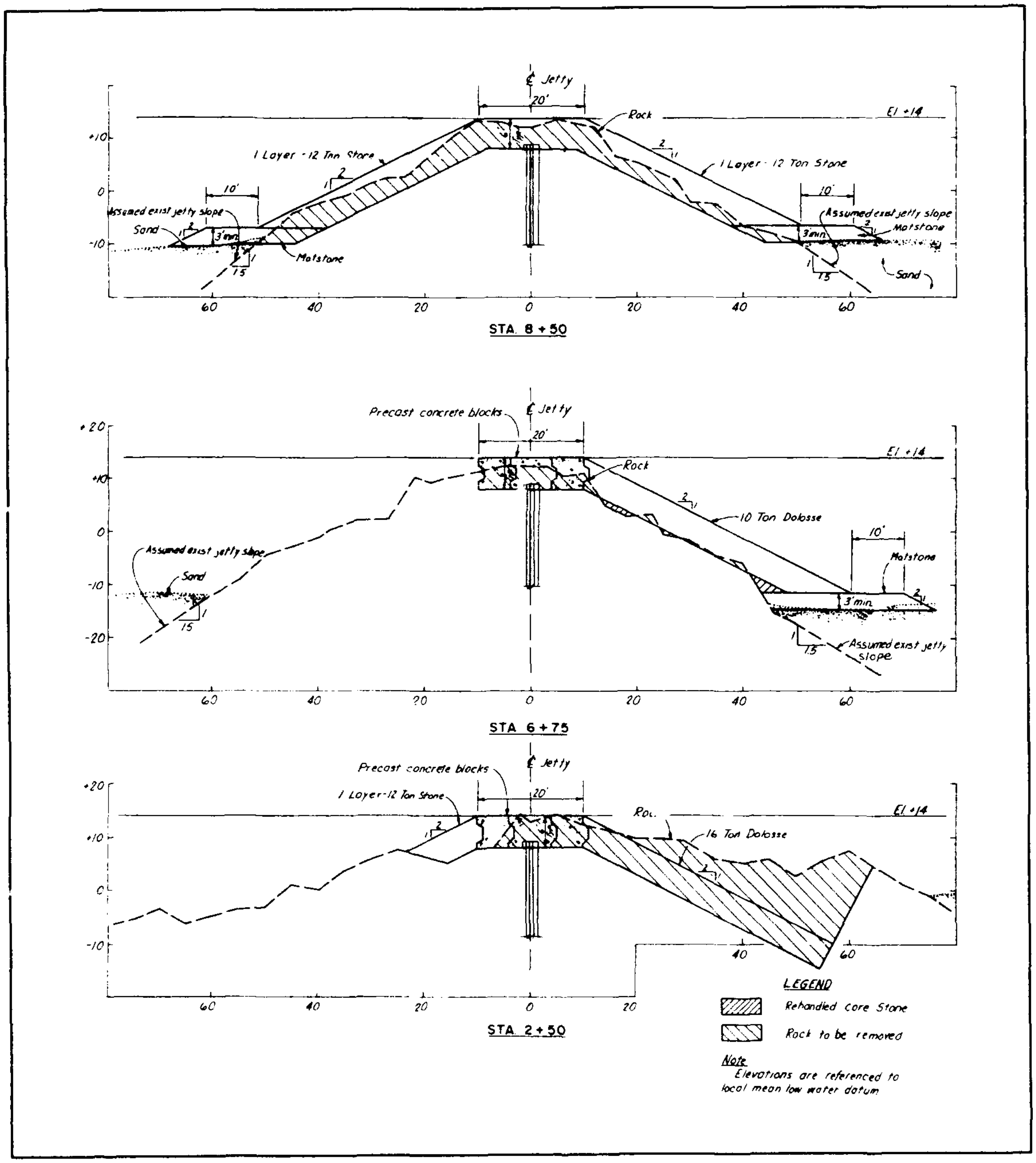


Figure 5. Construction drawings of north jetty cross sections

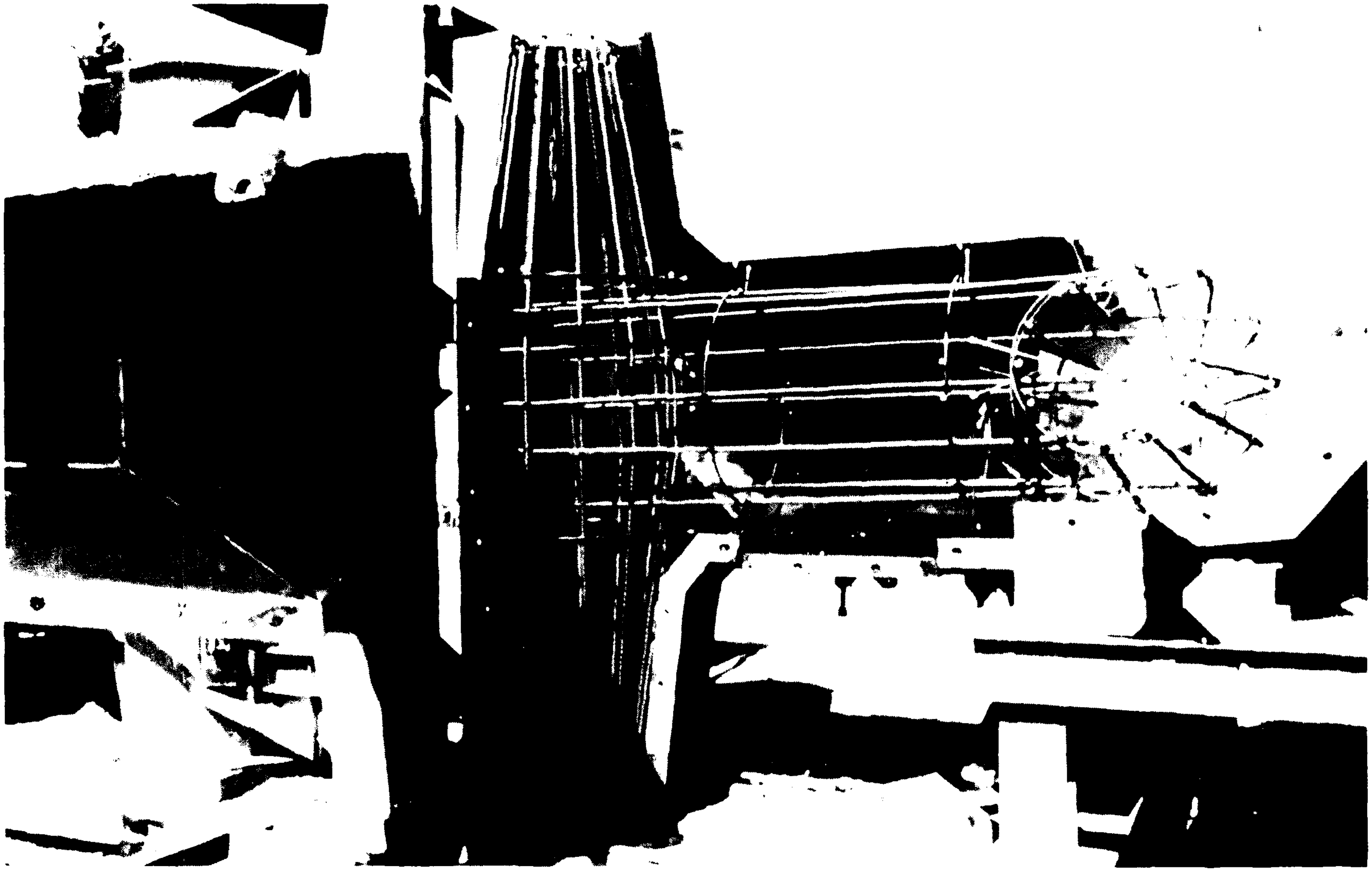


Figure 6. Epoxy coated, steel reinforcing rods used in Manasquan Inlet

Figure 7. View of inlet prior to rehabilitation of jetties with dolosse, January 1976

Figure 8. View of inlet after rehabilitation of jetties with dolosse, January 198[illegible]

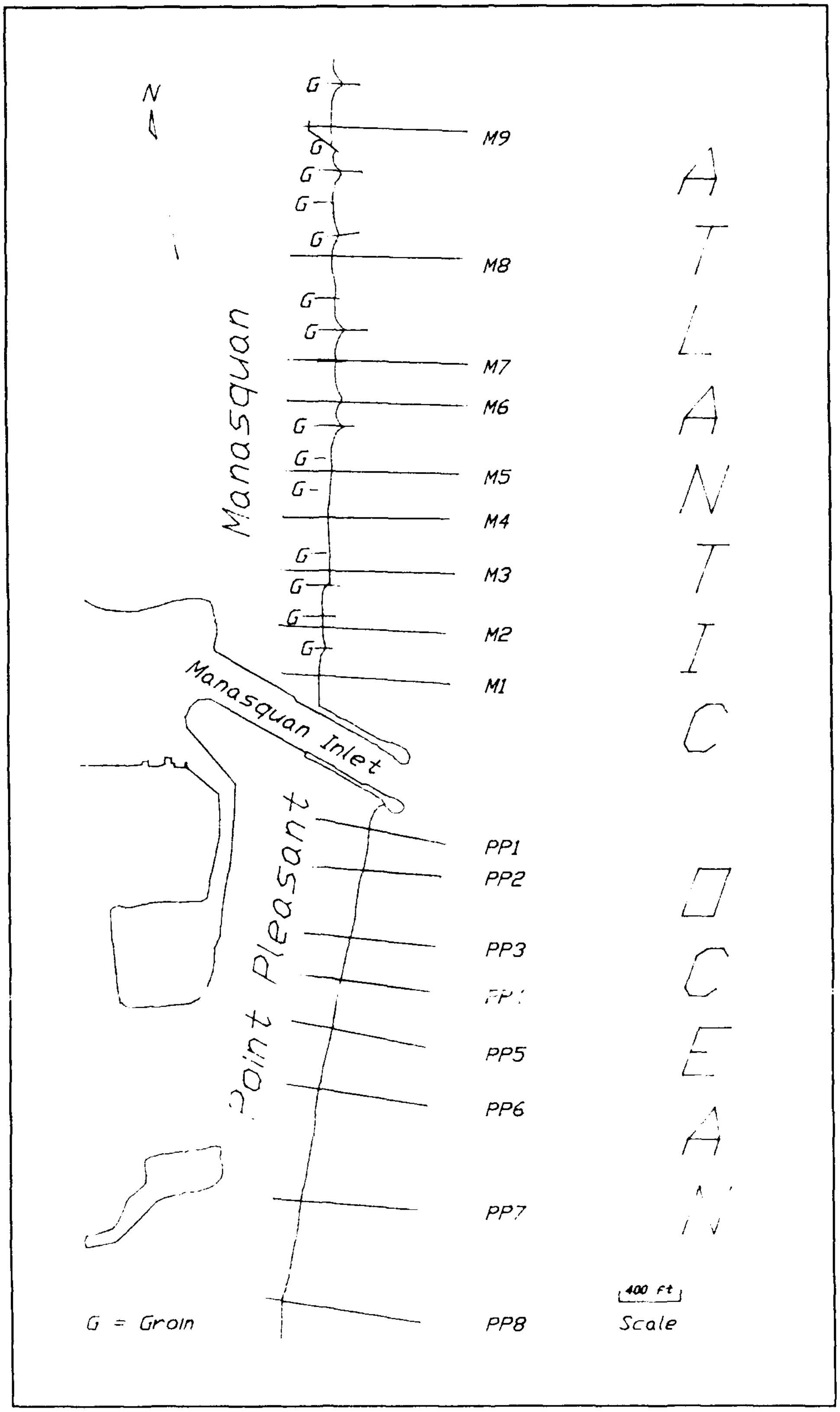


Figure 9. Location of beach profile lines

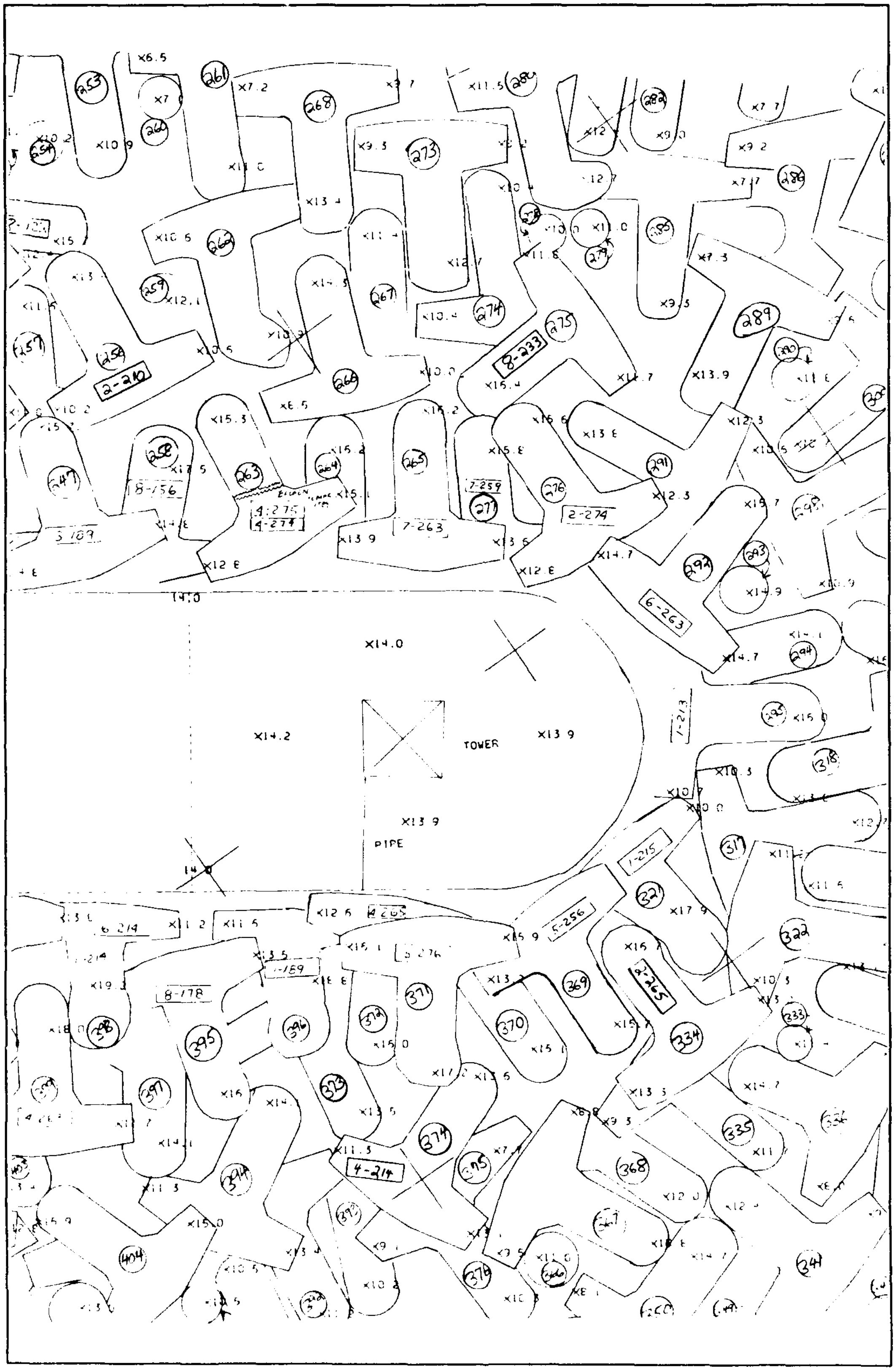


Figure 10. Portion of photogrammetric map of south jetty

LITTORAL ENVIRONMENT OBSERVATIONS (LEO)

(36001) MANASQUAN INLET, NEW JERSEY
LATITUDE 40 06.00 - LONGITUDE 74 02.00

DAY	TIME	WAVE PERIOD SEC	WAVE HEIGHT FEET	WAVE DIR	WAVE TYPE	WIND SPEED MPH	WIND DIR	SHORE SLOPE DEG	SURF WIDTH FEET	DYE DIST FEET	LONGSHORE CURRENT FT/SEC	LONGSHORE CURRENT DIR
1	0740	8.0	3.7	100	SP--PL	4	NW	**	140	****	*****	**
1	1630	8.5	1.8	110	PLUNGE	5	S	**	85	****	*****	**
2	0745	10.5	3.0	100	SP--PL	9	NW	**	125	****	*****	**
2	1620	9.0	3.2	100	SP--PL	11	NW	**	100	****	*****	**
3	1040	9.0	1.8	105	PLUNGE	11	S	**	85	****	*****	**
3	1705	7.0	1.9	105	SPILL	12	S	**	85	****	*****	**
4	0855	10.5	3.0	120	SP--PL	18	SW	**	165	****	*****	**
4	1645	8.0	2.9	100	SP--PL	10	W	**	150	****	*****	**
5	0650	7.5	2.5	105	SPILL	3	NW	**	90	****	*****	**
5	1900	7.5	3.2	115	SP--PL	16	S	**	125	****	*****	**
6	0825	6.5	1.5	100	SPILL	8	S	**	65	****	*****	**
6	1750	6.5	1.5	120	SP--PL	17	S	**	30	****	*****	**
7	0720	8.5	1.0	100	SPILL	7	W	**	60	****	*****	**
7	1815	7.5	2.8	100	SP--PL	3	SE	**	95	****	*****	**
8	0600	6.0	2.0	100	SP--PL	5	S	**	75	****	*****	**
8	1805	7.5	1.8	110	SPILL	13	S	**	60	****	*****	**
9	0935	5.0	3.5	075	SPILL	15	E	**	150	****	*****	**
9	1935	7.5	3.5	100	SP--PL	13	NE	**	180	****	*****	**
10	0720	6.0	3.5	080	SP--PL	10	NE	**	150	****	*****	**
10	1900	7.0	5.5	085	SP--PL	9	SE	**	250	****	*****	**
11	0920	9.0	3.5	085	SP--PL	5	W	**	140	****	*****	**
11	1950	8.0	3.8	095	SP--PL	11	SW	**	135	****	*****	**
12	0750	9.0	1.8	090	PLUNGE	7	W	**	80	****	*****	**
12	2005	8.5	3.0	095	SP--PL	6	SW	**	90	****	*****	**
13	0820	7.5	1.5	090	SURGE	6	N	**	50	****	*****	**
13	2000	10.0	1.0	095	SPILL	5	S	**	45	****	*****	**
14	0710	8.5	1.0	090	SURGE	2	SW	**	40	****	*****	**
14	1835	6.5	1.8	100	SPILL	12	S	**	45	****	*****	**
15	0525	8.0	1.3	095	PLUNGE	2	W	**	40	****	*****	**
15	1900	7.5	1.8	105	SPILL	11	S	**	50	****	*****	**
16	0740	8.5	1.5	105	SPILL	11	W	**	50	****	*****	**
16	1910	6.0	2.8	075	SP--PL	13	S	**	70	****	*****	**
17	0740	6.5	1.6	080	SPILL	3	NE	**	45	****	*****	**
17	1825	8.0	3.2	095	SP--PL	11	S	**	65	****	*****	**
18	0645	8.5	2.0	095	SP--PL	8	S	**	50	****	*****	**
18	2000	8.5	3.0	095	SP--PL	13	S	**	55	****	*****	**
19	0745	8.0	3.8	100	PLUNGE	7	SW	**	100	****	*****	**
19	1915	9.5	3.2	090	SP--PL	7	W	**	100	****	*****	**
20	0855	8.5	3.0	090	SP--PL	3	W	**	125	****	*****	**
20	1810	8.0	3.0	090	SPILL	11	NE	**	85	****	*****	**
21	2100	8.0	1.5	090	PLUNGE	3	N	**	40	****	*****	**
22	0945	6.5	1.0	085	SPILL	7	NE	**	35	****	*****	**
22	1855	7.5	1.3	090	SP--PL	4	S	**	45	****	*****	**
23	0800	7.0	1.5	090	PLUNGE	5	SE	**	50	****	*****	**
23	1935	7.0	1.8	110	SP--PL	10	SW	**	55	****	*****	**
24	0740	7.5	1.5	105	SPILL	8	W	**	45	****	*****	**
24	1920	6.5	1.1	100	SURGE	6	W	**	35	****	*****	**
25	0720	7.0	1.0	090	SPILL	12	NW	**	35	****	*****	**
25	1725	8.0	1.0	085	SPILL	13	NW	**	40	****	*****	**
26	0900	9.0	.5	090	SURGE	9	NE	**	35	****	*****	**
26	2000	6.5	1.5	110	SPILL	9	SW	**	40	****	*****	**
27	0730	9.0	.8	110	SPILL	12	W	**	30	****	*****	**
27	1850	5.5	2.3	110	SPILL	13	W	**	55	****	*****	**
28	0650	6.0	.8	110	SPILL	8	SW	**	35	****	*****	**
28	1930	7.0	4.8	075	SP--PL	23	NE	**	260	****	*****	**
29	0725	7.0	3.0	075	SP--PL	13	NE	**	150	****	*****	**
29	1845	5.0	3.0	085	SP--PL	13	S	**	115	****	*****	**
30	0725	6.0	2.5	090	SP--PL	6	SW	**	55	****	*****	**

Figure 11. Example of LEO data results

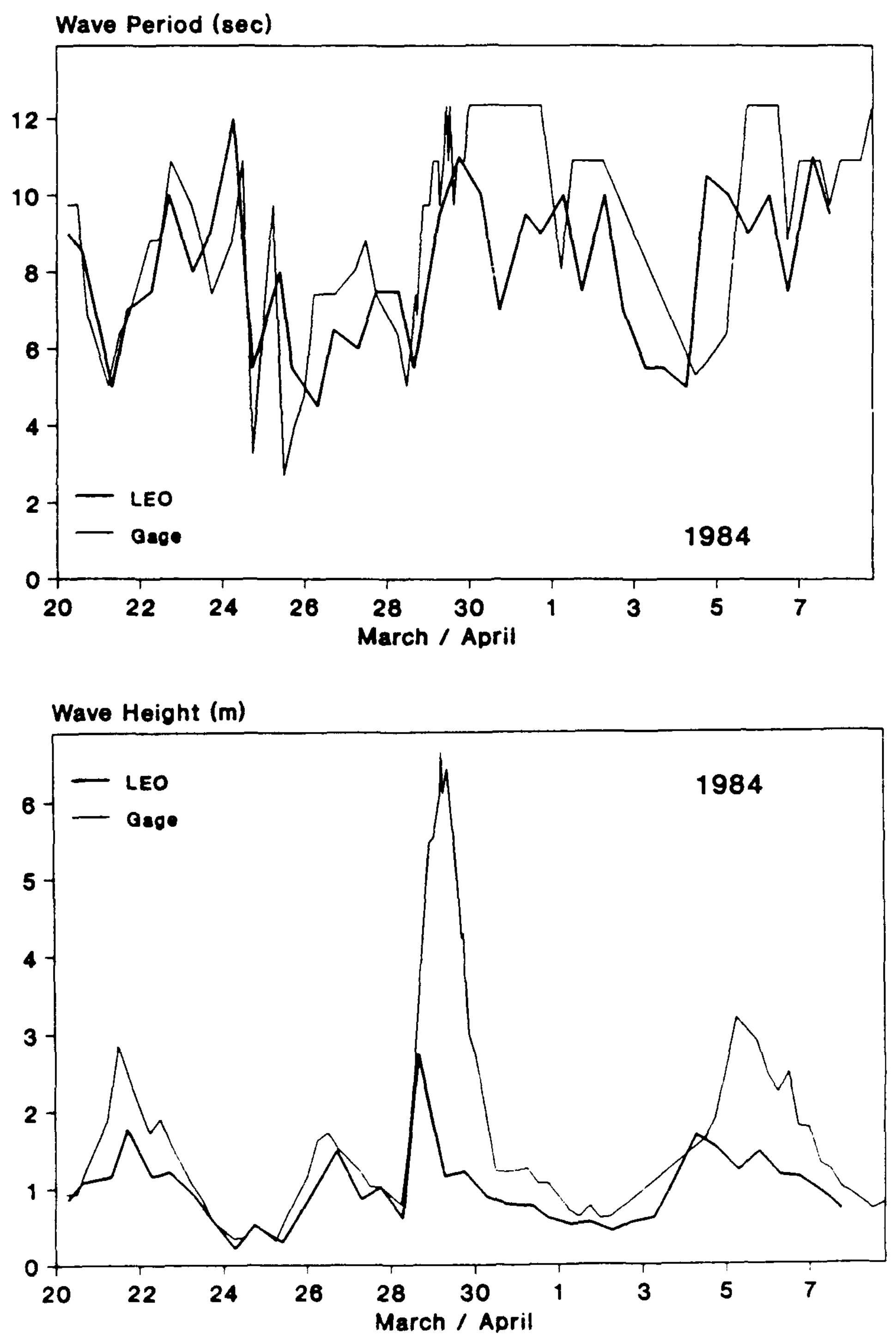


Figure 12. Wave period and height, LEO versus gage results, March/April 1984

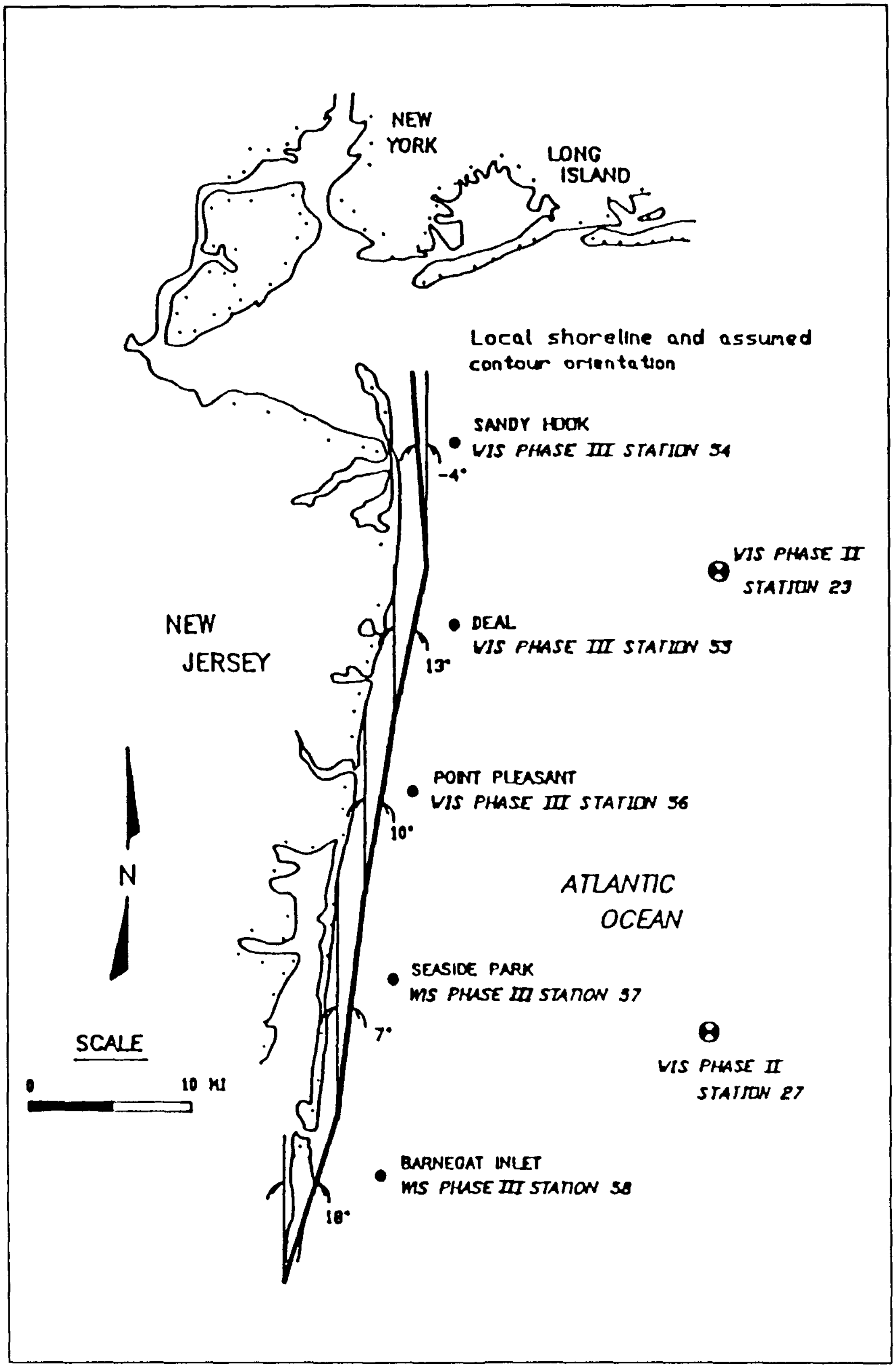


Figure 13. WIS Phase III hindcast stations

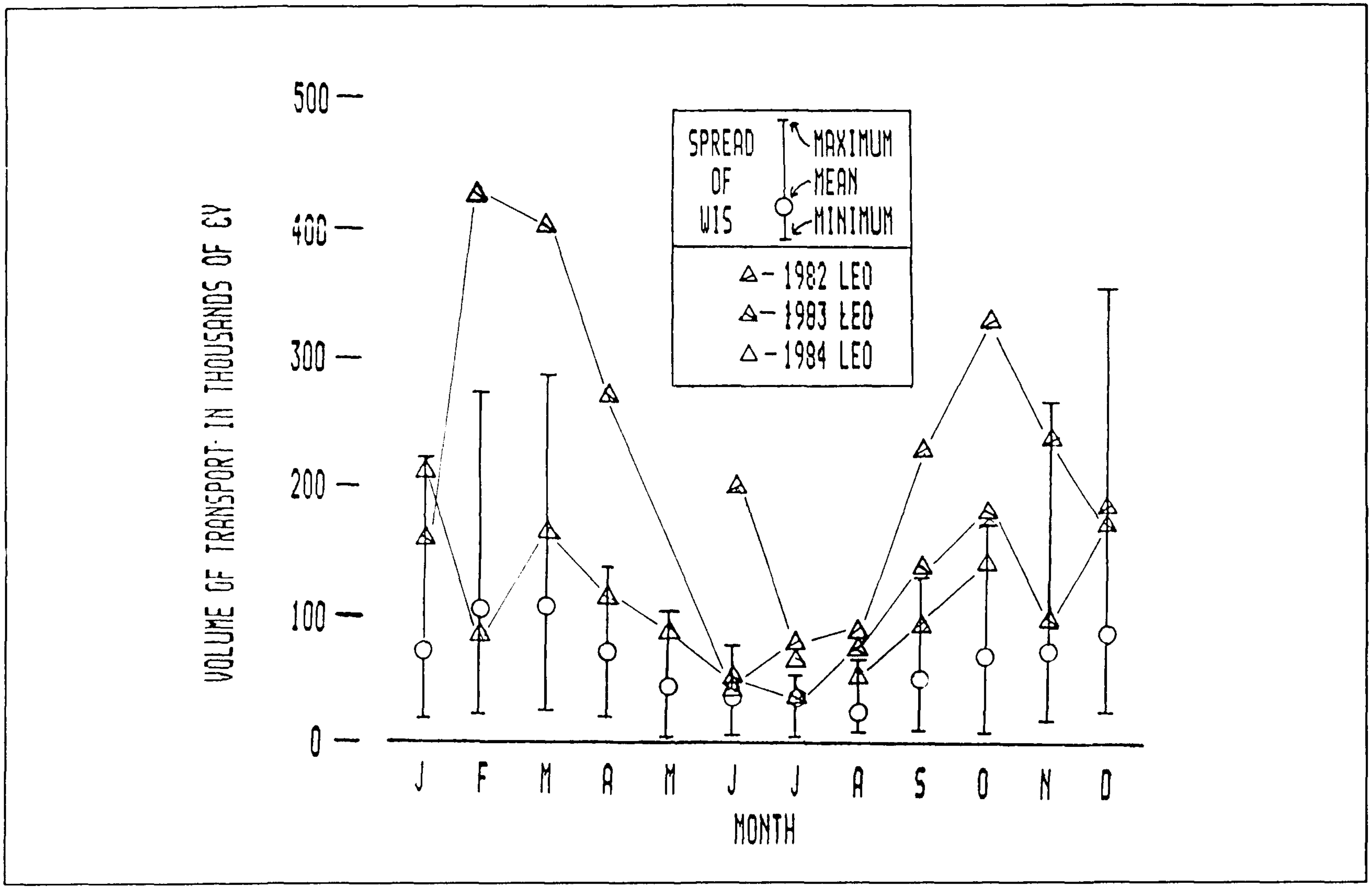


Figure 14. Comparison of LEO and WIS estimates of gross longshore transport

LITTORAL ENVIRONMENT OBSERVATIONS (LEO)
DATA SUMMARY

(35003) MANASQUAN INLET, NEW JERSEY
LATITUDE 40 06.00 - LONGITUDE 74 02.00

DATA COLLECTED FROM 15 FEB 82 TO 30 SEP 84

	JAN	FEB	MARCH	APRIL	MAY	JUNE	JULY	AUG	SEPT	OCT	NOV	DEC	TOTAL
FORESHORE SLOPE OBSERVATNS													
MAXIMUM SLOPE	0	0	9	9	0	9	0	9	20	0	0	0	20
MINIMUM SLOPE	0	0	9	9	0	9	0	9	20	0	0	0	9
AVERAGE SLOPE (2)	.0	.0	9.0	9.0	.0	9.0	.0	9.0	20.0	.0	.0	.0	11.4
NUMBER OF OBSERVATIONS	0	0	2	1	0	1	0	3	2	0	0	0	9
SEDIMENT TRANSPORT VOLUME (CUBIC YARDS X 1000)(4)													
METHOD 1													
NET CUBIC YARDS	126	203	206	1	-64	-31	-55	-14	78	183	-22	45	656
NUM OF OBSERVATIONS	115	113	122	121	62	164	181	154	180	119	119	101	1581
TOTAL LEFT CUBIC YDS	-28	-21	-39	-93	-72	-61	-59	-40	-38	-34	-94	-64	-643
NUM OF OBS TO LEFT	33	29	26	39	44	130	136	104	77	35	67	51	746
TOTAL RGHT CUBIC YDS	154	224	245	94	9	30	4	27	116	218	72	109	1302
NUM OF OBS TO RIGHT	54	59	54	40	4	32	15	39	65	61	31	30	494
METHOD 2													
NET CUBIC YARDS	0	0	0	0	0	0	0	0	0	0	0	0	0
NUM OF OBSERVATIONS	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL LEFT CUBIC YDS	0	0	0	0	0	0	0	0	0	0	0	0	0
NUM OF OBS TO LEFT	0	0	0	0	0	0	0	0	0	0	0	0	0
TOTAL RGHT CUBIC YDS	0	0	0	0	0	0	0	0	0	0	0	0	0
NUM OF OBS TO RIGHT	0	0	0	0	0	0	0	0	0	0	0	0	0

(1) CALMS, IF ANY, INCLUDED IN A[illegible]ATION
(2) CALMS NOT INCLUDED IN AVERA[illegible]
(3) A MINUS SIGN (-) INDICA[illegible]MENT TO THE LEFT
NO SIGN INDICATES CURRENT M[illegible] THE RIGHT
(4) ESTIMATED SEDIMENT TRANSPORT VOLUMES ARE GIVEN IN CUBIC YARDS. TWO METHODS (DESCRIBED IN SECTION 4 OF THE "SHORE PROTECTION MANUAL" (SPM)) ARE USED TO CALCULATE THE TRANSPORT VOLUME. NEGATIVE VALUES INDICATE TRANSPORT TO THE LEFT.
METHOD 1. THIS METHOD IS BASED ON EQUATIONS 4-38 AND 4-50B FROM THE SPM. A LONGSHORE ENERGY FLUX (EQUATION 4-38) IS FIRST CALCULATED FOR ONLY THE DAYS OF THE MONTH WHERE WAVE HEIGHT AND ANGLE OF APPROACH HAVE BEEN RECORDED, THEN AN AVERAGE FLUX FOR EACH MONTH IS CALCULATED, AND FINALLY THESE MONTHLY VALUES OF FLUX ARE SUBSTITUTED INTO EQUATION 4-50B AND DIVIDED BY 12 TO GET THE NET MONTHLY SEDIMENT TRANSPORT VOLUMES. THE YEARLY SEDIMENT TRANSPORT VOLUME IS CALCULATED BY SUMMING THE MONTHLY VALUES.
METHOD 2. THIS METHOD IS BASED ON EQUATIONS 4-51, 4-52, AND 4-50B FROM THE SPM, USING RECORDED OBSERVATIONS OF WAVE HEIGHT, WIDTH OF SURF ZONE, LONGSHORE CURRENT, AND DISTANCE TO DYE PATCH FROM SHORELINE AND FOLLOWING THE SAME PROCEDURE AS METHOD 1. NOTE: RECENT FINDINGS INDICATE A FRICTION FACTOR OF .006 SHOULD BE USED IN EQUATION 4-52.

Figure 15. Transport calculated as part of LEO ouptut (negative to the north)

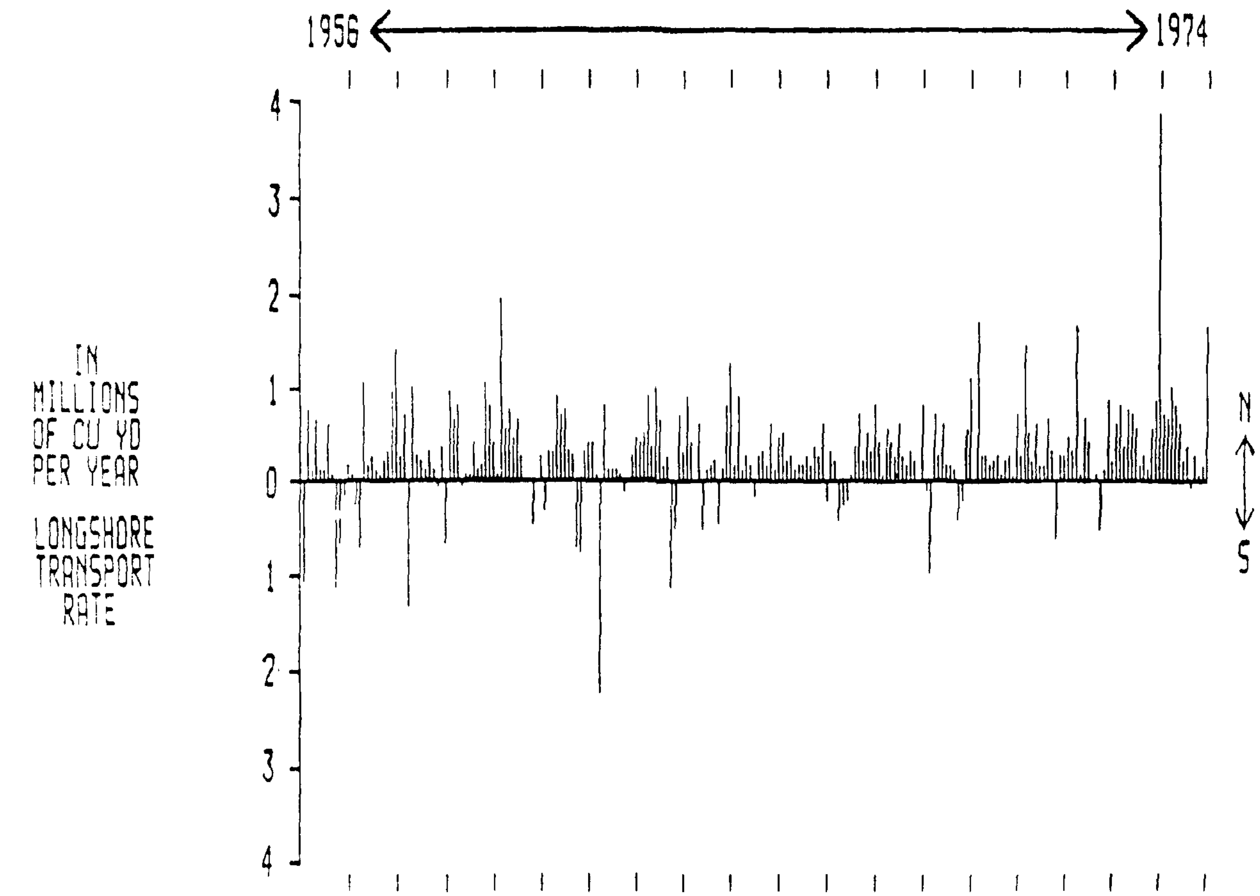


Figure 16. Monthly longshore transport rates for 1956-1974

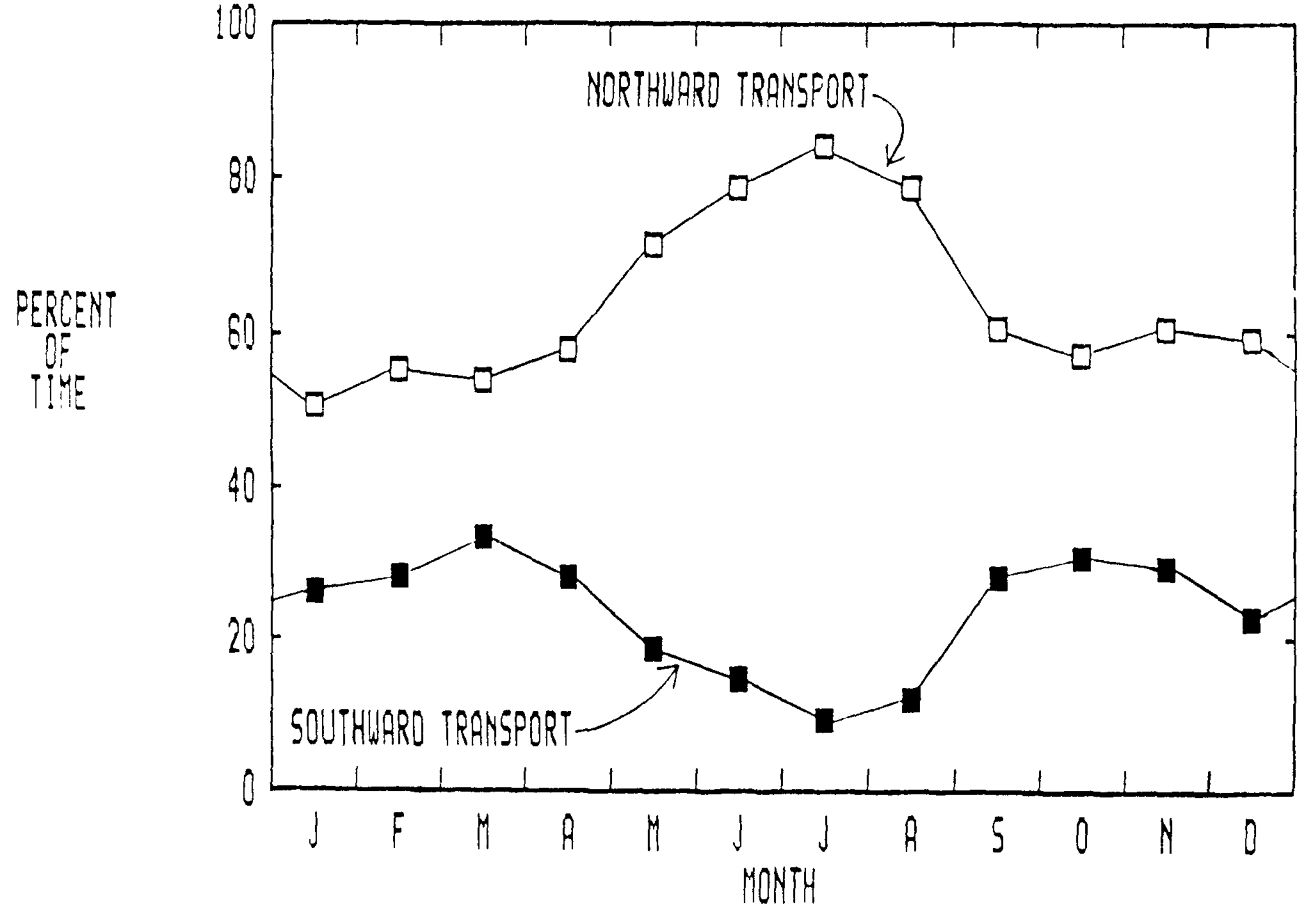


Figure 17. Seasonal variation in transport direction

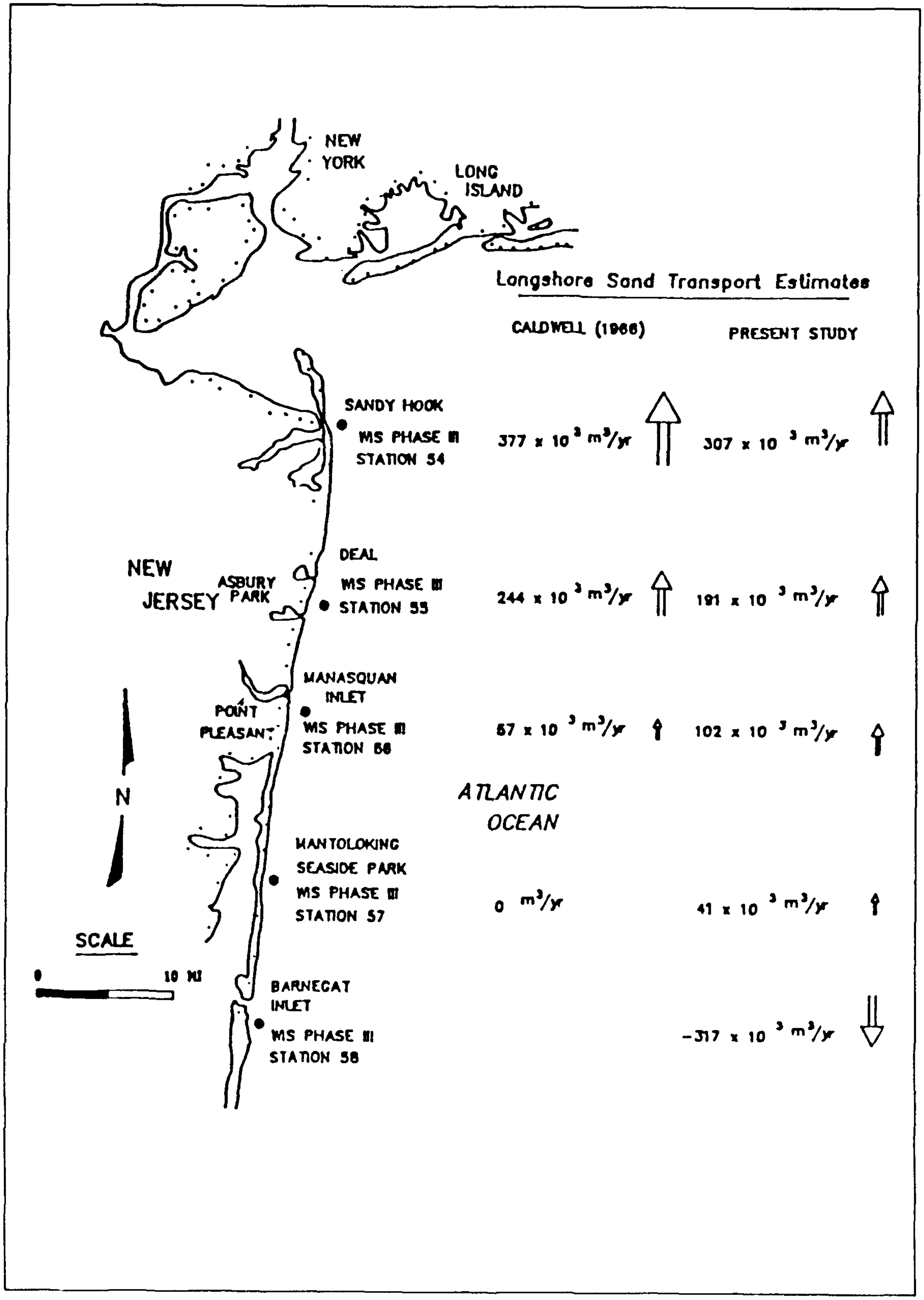


Figure 18. Potential longshore sand transport rates

I/J:	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
1 :	1	1	1	1	1	1	16	17	1	1	1	1	1	1	1	1	2	1	1	1	1	1	2	2	1
2 :	1	1	1	1	1	1	16	17	1	1	1	1	1	1	1	1	2	1	1	1	1	1	2	2	1
3 :	1	1	1	1	1	1	17	17	3	3	3	2	2	3	4	4	3	3	3	2	2	2	3	3	3
4 :	1	1	1	1	1	1	16	16	4	4	4	4	5	4	6	6	6	7	7	5	5	5	5	5	4
5 :	1	1	1	1	1	17	17	6	6	6	6	7	8	8	9	9	10	12	12	9	9	9	8	8	7
6 :	3	3	2	2	2	18	18	8	8	8	8	10	11	13	14	14	15	17	17	15	15	14	13	13	11
7 :	7	7	3	3	3	18	18	11	11	11	11	14	16	17	19	19	20	21	21	20	20	19	18	18	15
8 :	10	10	8	6	6	18	18	15	15	15	15	17	20	21	22	22	22	23	23	22	22	21	21	21	19
9 :	14	14	11	10	12	18	18	17	18	18	18	20	23	23	24	24	24	25	25	24	24	23	22	22	21
10 :	16	16	14	12	15	24	23	22	21	21	21	23	24	25	25	25	26	26	26	25	25	24	24	24	23
11 :	17	17	16	15	20	25	25	23	23	23	23	24	26	26	27	27	27	27	27	27	27	26	25	25	25
12 :	19	19	18	17	22	25	27	26	25	25	25	26	27	27	28	28	28	28	28	29	29	28	26	26	26
13 :	20	20	20	20	23	26	28	29	28	27	27	27	28	28	28	28	28	29	29	31	31	29	28	28	27
14 :	20	20	22	22	24	26	30	30	29	29	29	29	29	29	29	29	29	29	29	31	31	31	29	29	29
15 :	21	21	25	25	26	27	31	31	31	30	30	30	30	30	30	30	30	30	30	31	32	31	30	31	30
16 :	23	23	26	26	29	31	32	32	32	33	33	32	31	31	30	30	30	31	31	32	32	32	31	31	31
17 :	25	25	28	28	31	32	33	33	33	34	34	33	32	31	31	31	31	31	32	32	33	32	32	32	32
18 :	27	27	29	29	32	33	34	34	34	35	34	34	33	32	32	32	32	32	32	32	33	33	33	33	33
19 :	29	29	31	31	32	34	34	34	35	36	35	34	34	33	32	32	32	33	33	32	34	34	34	34	34
20 :	30	30	32	32	33	34	35	35	35	36	36	35	35	34	33	33	33	33	33	33	34	34	35	35	35
21 :	31	32	32	32	33	35	36	36	36	37	37	36	36	35	34	34	34	34	34	33	34	35	35	36	36
22 :	32	32	33	32	34	36	37	37	37	38	37	37	36	36	35	35	34	34	35	34	35	35	36	37	37
23 :	32	32	33	33	35	37	38	38	38	38	38	38	37	37	36	35	35	35	35	35	35	36	38	37	38
24 :	33	33	34	33	36	38	38	38	39	39	39	38	38	37	37	36	36	36	36	35	36	37	39	38	39
25 :	33	33	34	33	37	38	39	39	39	40	40	39	39	38	37	37	36	36	36	36	36	37	39	39	40
26 :	34	34	35	34	37	39	40	40	40	40	40	40	40	39	38	38	37	37	37	37	38	38	40	40	41
27 :	34	35	36	35	38	40	40	40	41	41	41	41	41	40	39	38	38	38	38	38	38	39	40	41	42
28 :	35	35	36	36	39	40	41	41	41	42	42	42	42	41	40	39	39	39	39	39	39	40	40	42	43
29 :	35	36	37	37	40	41	41	42	42	42	42	42	42	41	41	40	40	40	40	40	40	40	41	42	44
30 :	36	36	37	38	41	41	42	42	43	43	43	43	43	42	41	41	40	41	41	41	41	41	41	43	45
31 :	36	37	38	39	41	42	42	43	43	44	44	44	44	42	42	41	41	41	41	41	41	41	42	44	46
32 :	37	38	39	40	42	42	43	43	44	44	44	44	44	43	42	42	41	42	42	42	42	42	42	45	47
33 :	37	38	39	40	41	42	43	44	44	45	45	45	44	43	43	42	42	42	42	42	42	42	43	46	48

Figure 19. RCPWAVE depth grid, feet; lines represent jetties

I/J:	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
1 :	9	9	8	8	8	7	7	9	9	7	7	5	4	7	8	7	9	8	8	5	7	6	8	7	7
2 :	10	10	9	9	8	8	7	10	9	7	7	5	4	7	8	7	9	8	8	5	7	6	8	7	7
3 :	11	10	10	9	9	8	8	10	10	10	10	10	10	10	16	16	13	11	11	20	8	12	12	14	12
4 :	12	12	11	10	10	9	12	11	20	20	20	21	23	23	29	31	39	43	47	23	31	26	25	22	21
5 :	14	13	13	12	11	10	13	12	28	28	29	36	44	51	56	60	71	87	82	80	65	61	54	52	42
6 :	17	16	15	14	12	11	15	14	45	45	47	57	67	74	79	83	93	112	160	96	98	91	85	72	64
7 :	20	19	18	17	15	13	18	16	63	64	66	76	85	90	94	98	107	127	174	112	118	153	94	86	81
8 :	26	25	24	22	19	25	24	78	81	80	82	91	97	101	106	112	121	140	191	135	138	166	104	98	96
9 :	34	34	33	33	32	35	38	102	99	96	97	102	106	109	117	130	142	153	200	193	152	182	115	113	113
10 :	74	70	67	73	97	117	122	125	120	113	110	113	115	118	127	145	188	171	202	200	202	191	134	134	134
11 :	81	78	78	87	111	129	148	161	152	134	126	128	126	128	136	158	215	205	202	200	204	205	163	166	162
12 :	87	86	91	102	123	147	174	222	206	150	151	147	141	140	147	165	217	206	204	201	203	211	215	212	207
13 :	95	97	107	119	137	163	222	217	219	180	187	178	163	158	166	180	218	206	206	202	202	210	215	213	208
14 :	106	113	128	144	163	178	221	217	225	230	235	233	187	190	190	225	211	208	206	203	204	210	215	214	209
15 :	121	138	159	169	215	210	215	219	224	228	233	238	240	235	228	220	213	208	206	204	205	211	216	215	211
16 :	144	161	215	206	208	206	212	218	223	226	230	235	238	236	229	222	215	210	206	204	205	210	216	216	213
17 :	207	167	214	203	206	205	211	217	221	224	229	234	237	236	231	223	216	210	207	205	204	209	216	218	215
18 :	213	213	206	204	204	204	210	215	220	223	228	234	236	236	232	224	217	211	207	205	204	208	215	218	217
19 :	212	212	207	203	203	204	208	214	218	222	227	232	235	236	233	226	218	212	208	206	204	207	214	219	218
20 :	211	212	208	204	203	203	207	212	217	220	226	231	234	236	234	228	220	213	208	207	204	205	212	218	220
21 :	211	211	209	206	203	203	206	211	215	219	224	230	233	235	234	229	222	215	209	207	205	205	211	218	221
22 :	212	212	210	208	203	202	206	210	214	217	223	228	232	234	234	230	224	217	211	208	205	204	208	216	221
23 :	212	212	211	210	204	202	206	209	213	216	221	227	231	233	234	231	226	220	213	206	206	204	206	214	221
24 :	212	212	211	211	206	202	205	208	212	214	219	225	229	232	234	232	228	222	216	210	206	204	205	211	219
25 :	214	213	211	212	207	202	205	207	211	214	217	223	228	231	233	232	229	224	218	212	206	204	204	209	217
26 :	215	214	212	211	208	204	205	207	210	212	216	221	226	230	232	232	230	226	221	215	208	204	204	207	215
27 :	217	216	212	212	208	204	206	207	209	211	215	219	224	229	231	232	231	228	223	217	210	205	204	206	212
28 :	218	217	214	212	208	205	207	207	208	211	214	218	222	227	230	232	231	229	225	220	213	206	204	205	210
29 :	220	218	215	213	209	206	208	207	209	210	213	216	220	225	229	230	231	230	227	223	216	209	205	205	208
30 :	221	219	216	214	210	208	208	208	209	210	212	215	218	223	227	228	230	230	228	225	219	212	207	204	206
31 :	222	220	217	214	211	209	209	208	209	210	212	214	217	221	225	227	229	229	228	227	222	216	210	206	206
32 :	223	221	218	215	21[illegible]	210	209	208	209	210	212	214	216	220	223	225	227	228	228	227	224	219	213	208	207
33 :	224	222	219	216	213	212	210	209	209	210	212	213	215	219	221	223	226	226	226	227	225	221	216	210	208

Figure 20. Example of RCPWAVE wave heights; lines represent jetties

I/J:	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
1 :	4	5	8	5	2	-2	-22	-7	-19	6	8	7	10	15	12	10	8	4	3	4	7	10	9	5	3
2 :	4	5	9	6	2	-2	-25	-8	-22	6	8	7	10	15	12	10	8	4	3	4	7	10	9	5	3
3 :	4	4	10	7	2	-2	-29	-9	-25	12	13	11	13	19	19	14	12	9	5	5	8	10	9	8	10
4 :	4	4	8	8	3	-3	10	-10	4	16	19	21	18	18	21	19	20	17	11	10	12	11	11	11	11
5 :	4	3	6	9	4	-16	13	-18	15	18	22	25	24	23	22	23	25	23	17	15	16	15	15	14	14
6 :	7	4	6	18	6	-19	16	-16	18	20	24	29	28	28	27	27	29	27	23	22	21	20	20	19	19
7 :	14	9	8	18	11	-26	22	-22	21	23	27	31	31	31	31	31	31	30	27	26	25	24	25	24	23
8 :	20	19	18	23	23	23	23	21	24	25	29	33	33	33	33	33	33	32	30	28	27	27	27	27	26
9 :	25	25	24	31	36	23	23	23	26	27	30	33	34	34	34	34	35	34	32	30	29	28	29	29	29
10 :	28	29	28	32	38	37	27	26	26	28	30	33	34	34	35	35	36	35	33	31	30	29	30	30	30
11 :	30	32	31	34	40	36	30	28	28	29	30	33	34	35	36	36	36	36	35	33	31	30	30	31	32
12 :	32	34	33	34	39	36	33	30	29	30	31	32	34	35	36	37	37	37	37	35	32	31	31	32	32
13 :	33	36	35	35	37	36	34	32	31	31	31	32	34	35	36	37	37	38	38	37	34	32	32	33	33
14 :	34	37	37	35	36	36	35	33	32	32	32	33	34	35	37	37	38	38	38	38	35	34	33	34	34
15 :	34	38	39	37	37	35	35	34	33	32	32	33	34	36	37	38	38	38	38	38	36	35	34	35	35
16 :	36	38	39	38	39	37	35	35	35	34	33	33	34	36	37	38	38	39	39	38	37	36	35	35	35
17 :	37	39	40	39	39	37	36	35	35	35	34	34	35	36	37	38	39	39	39	39	38	36	36	36	36
18 :	39	40	41	39	39	37	36	35	35	35	34	34	35	36	37	39	39	39	39	39	39	37	36	36	36
19 :	40	41	41	40	40	38	37	36	36	35	35	35	35	36	38	39	39	40	40	40	39	38	37	36	37
20 :	41	41	42	41	40	39	37	36	36	36	35	35	36	37	38	39	40	40	40	40	40	38	37	37	37
21 :	42	42	42	41	41	39	38	37	37	36	36	36	36	37	38	39	40	40	40	40	41	39	38	37	37
22 :	42	42	42	42	41	39	38	37	37	37	36	36	37	37	38	40	40	41	41	41	41	40	38	37	38
23 :	42	42	42	42	42	40	39	38	38	37	36	37	37	38	39	40	40	41	41	41	41	41	39	38	38
24 :	43	42	42	42	43	40	39	38	38	38	37	37	37	38	39	40	41	41	42	42	42	41	40	38	38
25 :	43	43	42	42	44	41	40	39	39	38	38	37	38	38	39	40	41	42	42	42	42	42	41	39	38
26 :	44	43	43	42	43	41	40	40	39	39	38	38	38	39	39	40	41	42	42	43	43	42	41	40	39
27 :	44	44	43	43	43	42	41	40	39	39	38	38	38	39	40	41	42	42	43	43	43	43	42	41	39
28 :	44	44	44	43	44	42	41	41	40	39	39	39	39	39	40	41	42	43	43	44	44	44	42	41	39
29 :	44	44	44	44	44	42	42	41	40	40	39	39	39	39	40	41	42	43	44	44	44	44	43	42	40
30 :	45	44	44	44	44	42	42	42	41	40	40	39	39	39	40	41	42	43	43	44	45	45	44	43	40
31 :	45	45	45	44	44	43	43	42	41	41	40	40	40	40	41	41	42	43	43	44	45	45	45	44	41
32 :	45	45	45	45	44	43	43	42	42	41	41	40	40	40	41	41	42	43	43	44	45	46	46	45	42
33 :	45	45	45	44	44	44	43	42	42	41	41	41	40	40	41	41	42	43	43	44	45	46	46	45	43

Figure 21. Example of RCPWAVE wave angles; lines represent jetties

I/J:	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
1 :	102	102	102	102	102	102	26	25	89	89	89	93	102	93	86	86	81	86	93	93	93	86	76	76	86
2 :	102	102	102	102	102	102	26	25	89	89	89	93	102	93	86	86	81	86	93	93	93	86	76	76	86
3 :	102	102	102	102	102	102	26	26	63	63	63	70	76	65	54	54	57	61	61	70	70	66	61	61	61
4 :	102	102	102	102	102	102	26	26	51	51	51	49	47	50	43	43	41	39	39	45	45	46	47	47	49
5 :	102	102	102	102	102	25	25	42	42	42	42	39	37	36	35	35	32	30	30	34	34	35	37	37	39
6 :	60	60	72	72	72	25	25	37	37	37	37	33	31	29	28	28	27	26	26	27	27	28	29	29	32
7 :	40	40	59	59	59	25	25	31	31	31	31	28	26	25	24	24	24	23	23	24	24	24	25	25	27
8 :	32	32	37	41	42	25	25	27	27	27	27	25	24	23	23	23	22	22	22	23	23	23	23	23	24
9 :	28	28	32	33	30	25	25	26	25	25	25	23	22	22	22	22	22	21	21	22	22	22	22	22	23
10 :	26	26	28	30	27	22	22	23	23	23	23	22	22	21	21	21	21	21	21	21	21	22	22	22	22
11 :	25	25	26	27	24	21	21	22	22	22	22	22	21	21	21	21	21	21	21	21	21	21	21	21	21
12 :	24	24	25	25	23	21	21	21	21	21	21	21	21	20	20	20	20	20	20	20	20	20	21	21	21
13 :	24	24	23	23	22	21	20	20	20	20	20	20	20	20	20	20	20	20	20	19	19	20	20	20	20
14 :	23	23	23	23	22	21	20	20	20	20	20	20	20	20	20	20	20	20	20	19	19	19	20	20	20
15 :	23	23	21	21	21	21	19	19	19	19	19	20	20	20	20	20	20	20	20	19	19	19	19	19	20
16 :	22	22	21	21	20	19	19	19	19	19	19	19	19	19	20	20	20	19	19	19	19	19	19	19	19
17 :	21	21	20	20	19	19	19	19	19	19	19	19	19	19	19	19	19	19	19	19	19	19	19	19	19
18 :	20	20	20	20	19	19	19	19	19	18	18	19	19	19	19	19	19	19	19	19	19	19	19	19	19
19 :	20	20	19	19	19	19	18	18	18	18	18	18	19	19	19	19	19	19	19	19	19	19	19	19	18
20 :	20	20	19	19	19	18	18	18	18	18	18	18	18	19	19	19	19	19	19	19	18	18	18	18	18
21 :	19	19	19	19	19	18	18	18	18	18	18	18	18	18	18	19	19	19	18	19	18	18	18	18	18
22 :	19	19	19	19	18	18	18	18	18	18	18	18	18	18	18	18	18	18	18	19	18	18	18	18	18
23 :	19	19	19	19	18	18	18	18	18	18	18	18	18	18	18	18	18	18	18	18	18	18	18	18	18
24 :	19	19	19	19	18	18	18	18	18	17	17	18	18	18	18	18	18	18	18	18	18	18	17	18	17
25 :	19	19	18	19	18	18	17	17	17	17	17	17	17	18	18	18	18	18	18	18	18	18	17	17	17
26 :	18	18	18	18	18	17	17	17	17	17	17	17	17	17	18	18	18	18	18	18	18	18	17	17	17
27 :	18	18	18	18	18	17	17	17	17	17	17	17	17	17	17	18	18	18	18	18	18	17	17	17	17
28 :	18	18	18	18	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17
29 :	18	18	18	18	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17
30 :	18	18	18	18	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	16
31 :	18	18	18	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	16
32 :	18	18	18	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	17	16	16
33 :	18	18	17	17	17	17	17	17	17	16	16	16	17	17	17	17	17	17	17	17	17	17	17	16	16

Figure 22. Example of RCPWAVE wave number; lines represent jetties

I/J:	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
1 :	11	11	11	11	11	11	0	0	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11
2 :	11	11	11	11	11	11	0	0	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11
3 :	11	11	11	11	11	11	0	0	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11
4 :	11	11	11	11	11	11	0	0	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11
5 :	11	11	11	11	11	0	0	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11
6 :	11	11	11	11	11	11	0	11	11	11	11	11	11	11	11	11	11	11	0	11	11	11	11	11	11
7 :	11	11	11	11	11	11	0	11	11	11	11	11	11	11	11	11	11	11	0	11	11	0	11	11	11
8 :	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	0	11	11	0	11	11	11
9 :	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	0	11	11	0	11	11	11
10 :	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	0	0	0	0	11	11	11
11 :	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	0	0	0	0	0	0	11	11	11
12 :	11	11	11	11	11	11	11	0	0	11	11	11	11	11	11	11	0	0	0	0	0	0	0	0	0
13 :	11	11	11	11	11	11	0	0	0	11	11	11	11	11	11	11	0	0	0	0	0	0	0	0	0
14 :	11	11	11	11	11	11	0	0	0	0	0	11	11	11	11	0	0	0	0	0	0	0	0	0	0
15 :	11	11	11	11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
16 :	11	11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
17 :	0	11	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
18 :	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
19 :	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
20 :	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
21 :	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
22 :	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
23 :	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
24 :	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
25 :	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
26 :	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
27 :	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
28 :	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
29 :	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
30 :	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
31 :	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
32 :	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
33 :	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Figure 23. Example of RCPWAVE breaker indexes; lines represent jetties

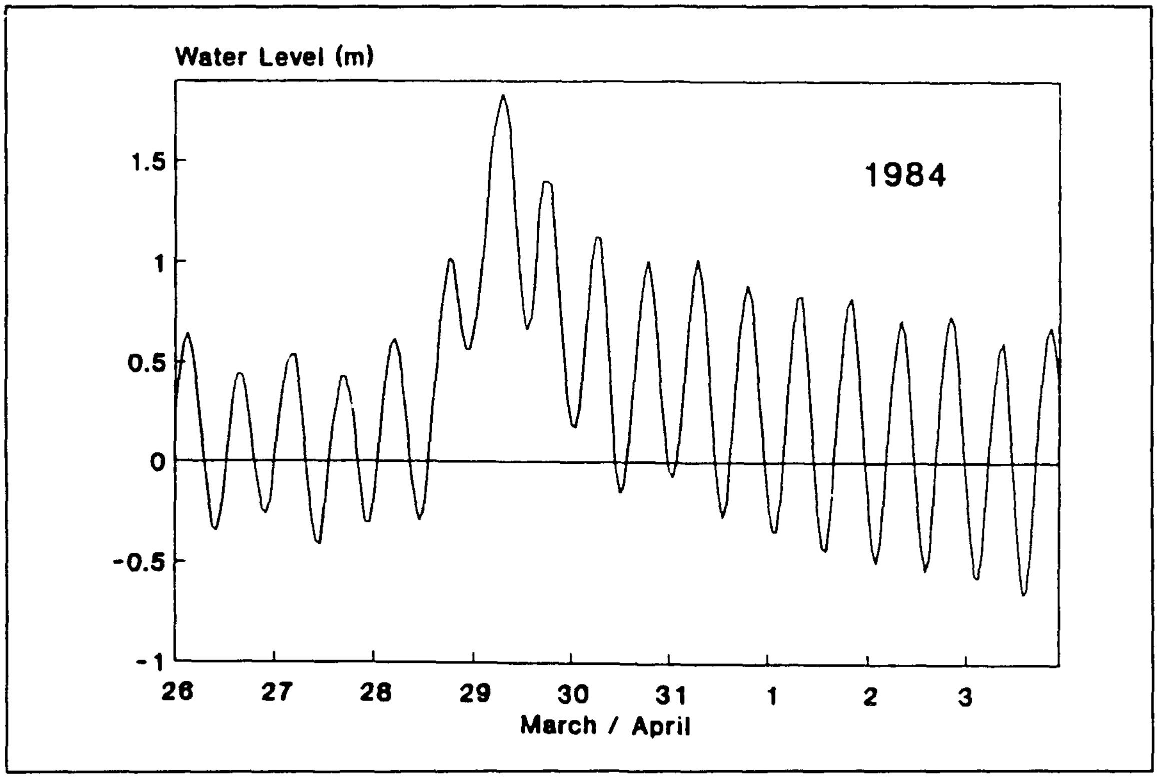


Figure 24. Wave level measured during March/April 1984 storm

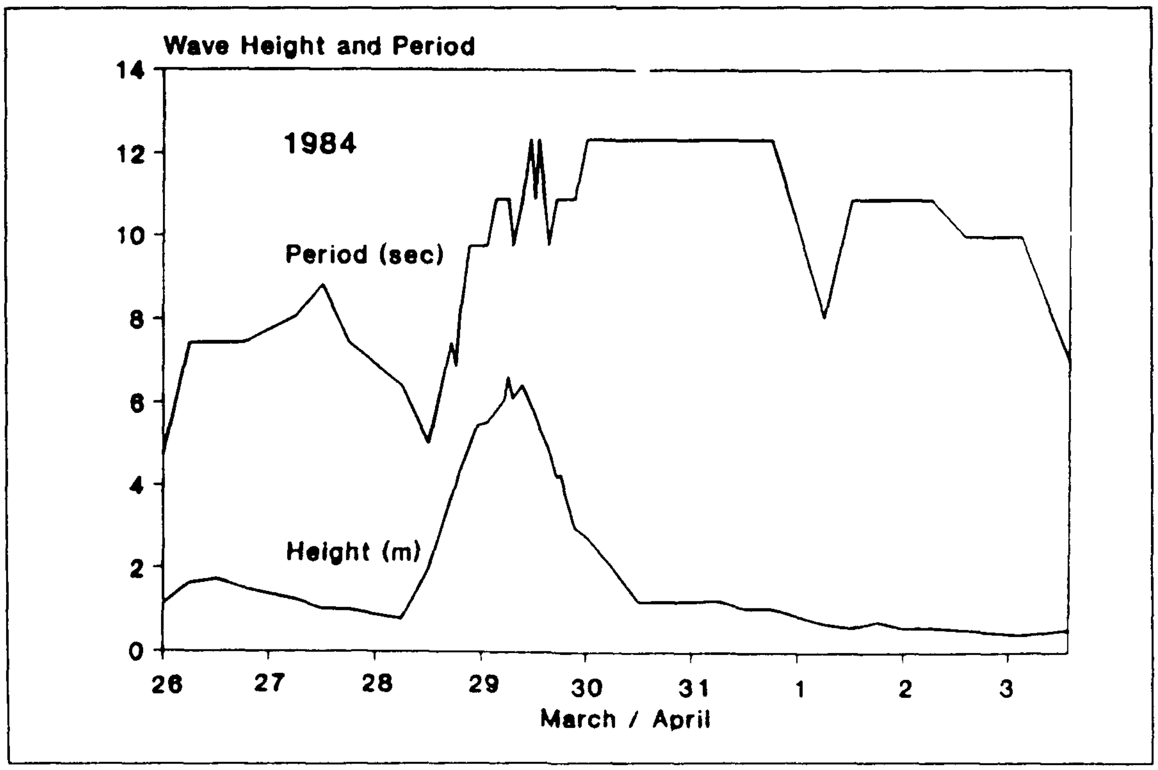


Figure 25. Wave height and period measured during March/April 1984 storm

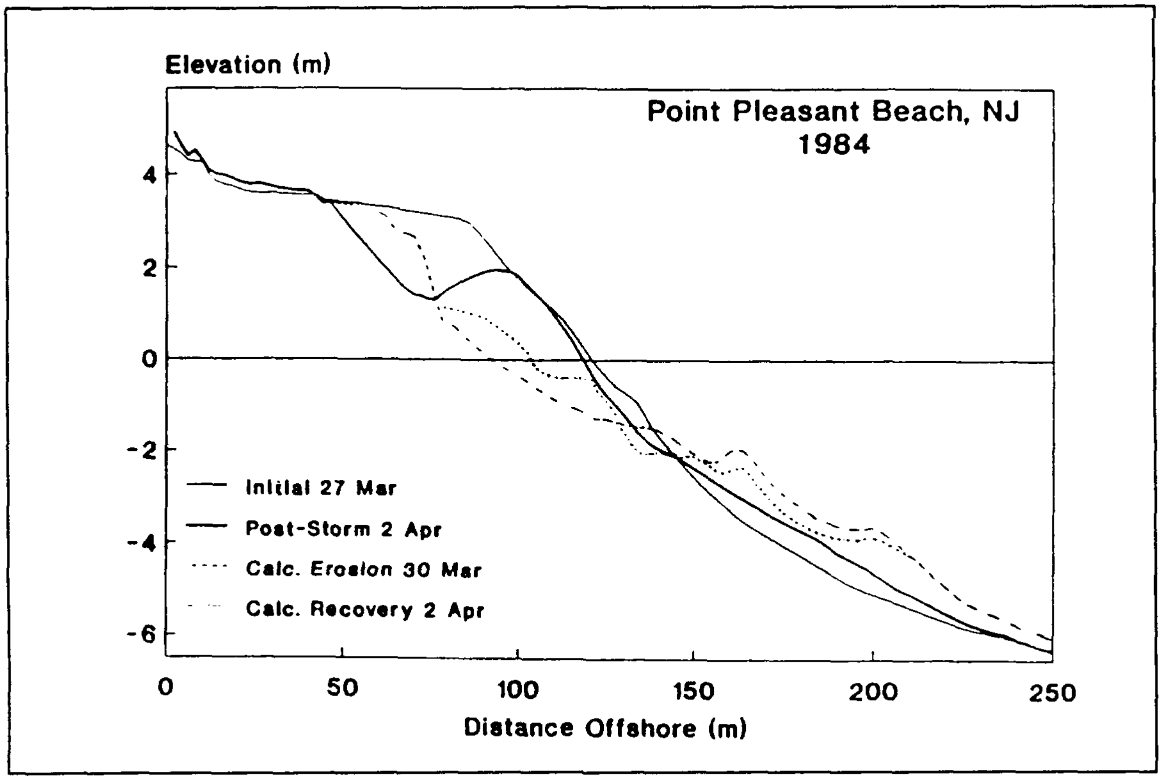


Figure 26. Storm-induced profile changes at Point Pleasant Beach

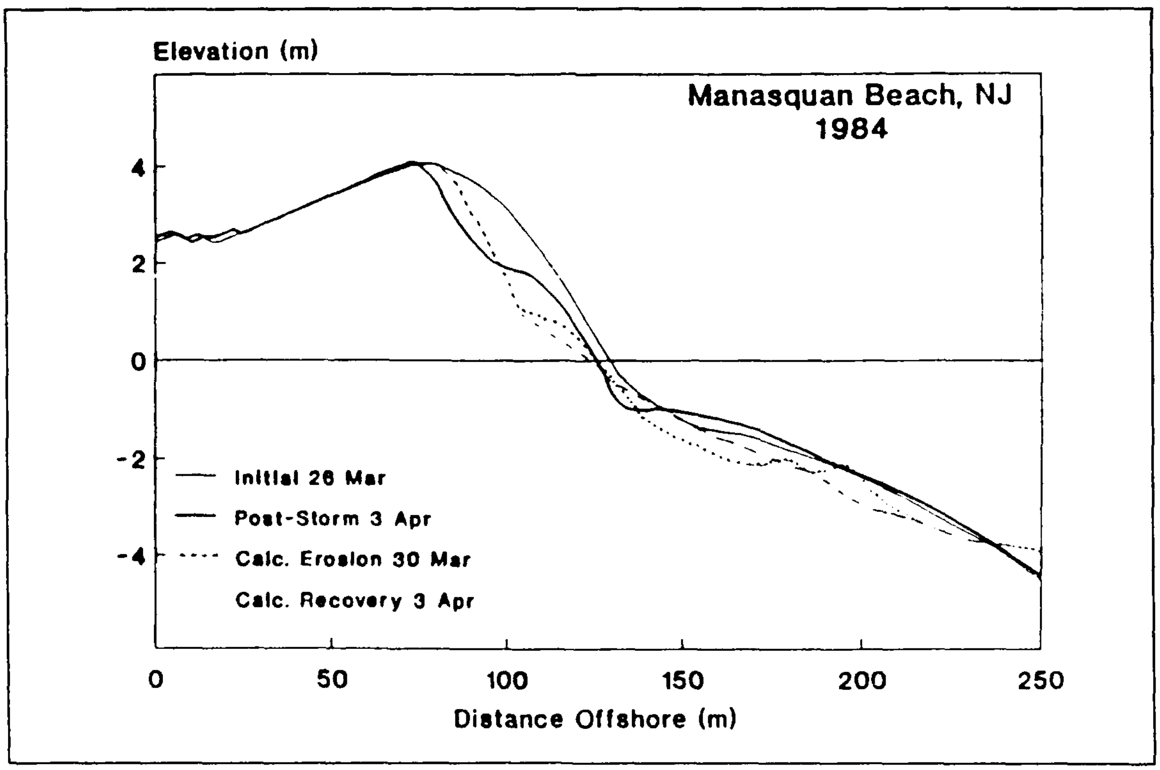


Figure 27. Storm-induced profile changes at Manasquan Beach

NATIONAL OCEAN SURVEY (NOAA)
TIDES HIGH AND LOW WATERS (FEET)
TM 075W M

8532591 MANASQUAN INLET, N.J.
JUN, 1983

DAY	HIGH TIME	HIGH HEIGHT	LOW TIME	LOW HEIGHT
1	11.3	5.43	> 5.3	2.65
	> 23.7	5.97	17.2	3.22
2	12.3	5.10	> 5.8	2.52
			18.4	3.07
3	> 0.3	5.59	> 6.8	2.35
	13.3	5.27	19.1	3.22
4	> 1.5	5.81	> 8.1	2.43
	14.1	5.12	20.3	3.04
5	> 2.0	5.58	> 8.2	2.41
	> 14.8	5.70	21.3	2.89
6	2.8	5.32	> 9.4	2.21
	> 15.5	5.95	22.0	2.71
7	3.8	5.61	> 10.3	2.09
	> 16.5	6.19	22.9	2.34
8	4.8	5.78	10.9	2.05
	> 17.2	6.55	> 23.8	1.83
9	5.7	5.76	11.6	1.96
	> 18.1	6.80		
10	6.5	5.99	> 0.4	1.63
	> 18.9	6.98	12.6	1.66

DAY	HIGH TIME	HIGH HEIGHT	LOW TIME	LOW HEIGHT
11	7.5	5.72	> 1.3	1.34
	> 19.7	7.02	13.3	1.35
12	8.3	5.81	> 2.1	1.06
	> 20.6	7.17	14.0	1.30
13	9.2	6.07	> 2.8	1.20
	> 21.5	7.26	14.9	1.59
14	9.9	6.18	> 3.8	1.51
	> 22.5	7.07	16.1	1.84
15	10.9	6.12	> 4.9	1.43
	> 23.1	6.78	16.8	2.07
16	11.9	6.21	> 5.7	1.68
			17.8	2.36
17	> 0.4	6.55	> 6.7	1.75
	13.0	6.10	19.1	2.43
18	> 1.4	6.35	> 7.8	1.86
	14.3	6.24	20.3	2.41
19	2.5	5.94	> 8.7	1.89
	> 15.3	6.26	21.5	2.13
20	3.3	5.69	> 9.7	1.93
	> 16.2	6.48	22.3	2.29

DAY	HIGH TIME	HIGH HEIGHT	LOW TIME	LOW HEIGHT
21	4.2	5.66	> 10.5	2.05
	> 17.0	6.58	23.0	2.10
22	5.4	5.62	> 11.2	1.92
	> 17.8	6.47		
23	6.0	5.48	> 0.2	1.75
	> 18.5	6.42	12.2	1.83
24	6.8	5.21	> 0.8	1.58
	> 19.2	6.50	12.7	1.94
25	7.8	5.64	> 1.5	1.82
	> 19.9	6.72	13.2	2.26
26	8.2	5.62	> 2.3	1.95
	> 20.4	6.54	14.1	2.42
27	9.0	5.40	> 2.8	1.89
	> 21.1	6.17	14.6	2.39
28	9.7	5.55	> 3.4	1.95
	> 21.9	7.01	15.5	3.01
29	10.3	5.96	> 4.1	2.93
	> 22.4	6.26	16.1	3.03
30	11.0	5.42	> 4.9	2.17
	> 23.1	5.90	16.8	2.77

MEANS FOR JUN 1, 1983 - JUN 30, 1983

MHW	6.06	MTL-MSL	-0.02	GT	4.51	
MLW	2.13	MHHW	6.44	(DRL)TL	4.10	
MSL	4.11	MLLW	1.92	(DRL)TL-MSL	0.07	
MN	3.94	DHQ	0.37	GMHWI	0.05	HRS
MTL	4.10	DLQ	0.21	GMLWI	6.23	HRS

KEY
> HIGHER HIGH/LOWER LOW

HIGHEST TIDE 7.26 21.5 HRS JUN 13, 1983
LOWEST TIDE 1.06 2.1 HRS JUN 12, 1983

ARCHIVED	TID	EID	VID	START	STOP	SETTING	CNST	KPT
83 6 10	232	0	204	83 5 1 10	83 6 1 0	-1016	0	646
83 0 21	232	232	211	83 6 1 0	83 7 2 10	-1016	0	594

Figure 28. Time and height of high and low tides

NATIONAL OCEAN SURVEY (NOAA)
TIDES, HOURLY HEIGHTS (FEET)

8532591 MANASQUAN INLET, N.J. TM 075W M

JUN. 1983	0/12	1/13	2/14	3/15	4/16	5/17	6/18	7/19	8/20	9/21	10/22	11/23
1	5.98	5.32	4.51	3.70	3.01	2.70	2.83	3.24	3.60	[4.50]	4.93	5.35
	5.30	4.88	4.41	3.83	3.48	3.28	3.25	3.75	4.10	4.98	5.35	5.82
2	5.34	5.37	4.65	3.73	3.06	2.61	2.53	2.60	3.03	3.70	4.22	4.77
	5.12	5.14	4.63	4.09	3.64	3.33	3.09	3.10	3.59	4.24	4.81	5.28
3	5.57	5.49	4.99	4.33	3.65	2.98	2.53	2.35	2.60	3.25	3.94	4.56
	5.03	5.24	5.08	4.71	4.20	3.79	3.40	3.23	3.32	3.66	4.23	4.83
4	5.32	5.76	5.66	5.19	4.51	3.85	3.15	2.60	2.38	2.81	3.45	4.12
	4.62	4.96	5.07	5.03	4.64	4.38	3.86	3.38	3.06	3.18	3.54	4.24
5	4.90	5.33	5.56	5.25	4.73	3.92	3.41	2.73	2.46	2.48	2.81	3.55
	4.32	5.01	5.47	5.70	5.36	4.81	4.28	3.71	3.18	2.91	3.04	3.46
6	4.11	4.72	5.12	5.35	5.16	4.59	3.87	3.14	2.58	2.23	2.31	2.89
	3.87	4.76	5.40	5.84	5.90	5.62	4.39	4.34	3.70	2.99	2.72	2.97
7	3.44	4.30	4.94	5.55	5.63	5.31	4.52	3.76	3.04	2.37	2.15	2.30
	2.89	3.36	4.93	5.61	6.06	6.13	5.54	4.73	4.05	3.16	2.54	2.33
8	2.71	3.31	4.29	5.03	5.57	5.79	5.39	4.57	3.70	2.78	2.28	2.06
	2.39	3.19	4.40	5.27	6.12	6.51	6.44	5.58	4.62	3.62	2.70	2.01
9	1.89	2.42	3.13	4.11	4.84	5.50	5.70	5.26	4.45	3.48	2.75	2.13
	1.97	2.41	3.58	4.62	5.61	6.34	6.64	6.52	5.62	4.46	3.25	2.41
10	1.71	1.81	2.55	3.48	4.43	5.30	5.66	5.79	5.19	4.17	3.18	2.39
	1.78	1.71	2.55	3.84	5.11	6.02	6.80	6.96	6.42	5.48	4.28	3.13
11	2.06	1.38	1.60	2.29	3.30	4.39	5.19	5.61	5.56	4.84	3.86	2.78
	1.87	1.40	1.62	2.64	3.99	5.26	6.18	6.88	6.94	6.31	5.21	3.93
12	2.68	1.61	1.05	1.36	2.25	3.21	4.33	5.30	5.77	5.51	4.70	3.68
	2.64	1.73	1.30	1.78	2.82	4.20	5.46	6.38	7.03	7.06	6.27	5.07
13	3.72	2.58	1.56	1.22	1.70	2.56	3.63	4.74	5.60	6.05	5.61	4.67
	3.74	2.79	1.93	1.59	2.11	3.20	4.50	5.64	6.56	7.11	7.15	6.29
14	5.09	3.78	2.64	1.76	1.53	2.14	2.94	3.99	4.94	5.79	6.17	5.61
	4.91	3.88	2.94	2.09	1.66	2.33	3.35	4.55	5.60	6.49	7.01	6.69
15	5.94	4.75	3.60	2.55	1.70	1.46	2.07	2.93	3.93	4.92	5.77	6.12
	5.65	4.90	3.92	3.00	2.26	2.10	2.52	3.40	4.53	5.47	6.21	6.60
16	6.49	5.73	4.59	3.61	2.59	1.64	1.70	2.25	3.14	4.14	5.13	5.88
	6.21	5.80	5.16	4.18	3.27	2.60	2.37	2.66	3.51	4.49	5.29	6.01
17	6.51	6.42	5.62	4.55	3.52	2.65	1.95	1.60	2.35	3.21	4.19	5.06
	5.81	6.11	5.85	5.17	4.35	3.56	2.84	2.43	2.65	3.30	4.29	5.08
18	5.85	6.30	6.18	5.41	4.43	3.54	2.65	2.09	1.91	2.41	3.26	4.28
	5.17	5.84	6.22	6.02	5.29	4.49	3.63	2.86	2.45	2.62	3.17	4.00
19	4.81	5.51	5.87	5.89	5.20	4.27	3.48	2.64	2.09	1.94	2.49	3.39
	4.40	5.20	5.93	6.22	5.98	5.45	4.56	3.71	2.78	2.29	2.38	2.99
20	3.84	4.72	5.31	5.66	5.55	4.93	4.18	3.38	2.67	2.10	1.96	2.52
	3.46	4.48	5.36	6.12	6.51	6.28	5.51	4.59	3.58	2.72	2.30	2.47
21	3.09	3.92	4.70	5.30	5.67	5.58	4.94	4.12	3.27	2.57	2.08	2.16
	2.81	3.74	4.81	5.63	6.26	6.59	6.16	5.32	4.28	3.30	2.51	2.10
22	2.30	2.97	3.77	4.59	5.24	5.53	5.51	4.85	3.91	3.09	2.39	1.95
	2.11	2.87	3.89	4.91	5.68	6.30	6.46	5.98	4.95	3.90	3.00	2.14
23	1.76	2.10	2.74	3.60	4.42	5.08	5.47	5.23	4.40	3.59	2.75	2.10
	1.86	2.12	2.99	4.11	5.10	5.87	6.40	6.36	5.60	4.57	3.56	2.52

NATIONAL OCEAN SURVEY (NOAA)
TIDES, HOURLY HEIGHTS (FEET)

8532591 MANASQUAN INLET, N.J. TM 075W M

JUN. 1983	0/12	1/13	2/14	3/15	4/16	5/17	6/18	7/19	8/20	9/21	10/22	11/23
24	1.79	1.60	2.01	2.77	3.64	4.43	5.05	5.21	4.78	4.08	3.29	2.61
	2.08	1.95	2.44	3.43	4.50	5.39	6.13	6.51	6.26	5.46	4.45	3.45
25	2.54	1.93	1.67	2.52	3.34	4.22	4.93	5.48	5.61	5.09	4.31	3.42
	2.67	2.28	2.48	3.15	4.07	5.05	5.64	6.54	6.72	6.20	5.31	4.29
26	3.30	2.45	1.98	2.24	2.90	3.73	4.58	5.27	5.60	5.40	4.76	4.04
	3.32	2.69	2.38	2.64	3.53	4.56	5.44	6.11	6.48	6.34	5.74	4.78
27	3.80	2.84	2.14	1.92	2.30	2.99	3.86	4.67	5.22	5.37	5.06	4.33
	3.61	2.98	2.51	2.44	2.76	3.63	4.57	5.34	5.86	6.16	5.84	5.01
28	4.05	3.16	2.39	1.98	2.09	2.62	3.39	4.18	4.62	5.33	5.53	5.12
	4.46	3.94	3.49	3.09	3.16	3.76	4.69	5.65	6.33	6.65	6.99	6.59
29	5.66	4.94	3.88	3.33	2.88	2.99	3.61	4.40	4.97	5.64	5.89	5.66
	5.18	4.45	3.80	3.25	3.02	3.29	3.89	4.71	5.42	5.93	6.24	6.16
30	5.51	4.69	3.72	3.02	2.37	2.18	2.60	3.35	4.06	4.74	5.27	5.42
	5.12	4.53	3.92	3.34	2.96	2.82	3.13	3.84	4.61	5.27	5.71	5.68

MSL= 4.11

KEY
[ ] INFERRED TIDE

ARCHIVED	TID	EID	VID	START	STOP	SETTING	CNST	RPT
83 6 18	232	0	204	83 5 1 10	83 6 1 8	-1016	0	646
83 8 21	232	232	211	83 6 1 8	83 7 2 10	-1016	0	594

DATUM IS ....

Figure 29. Hourly tide heights

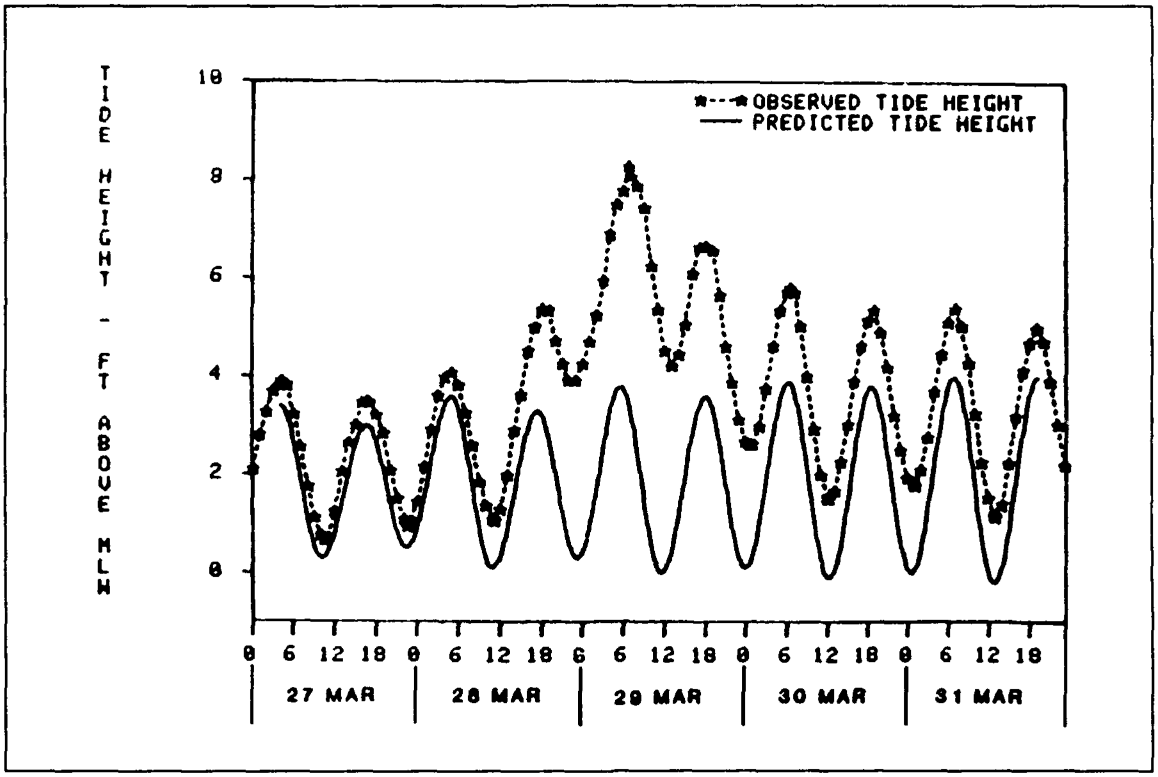


Figure 30. Observed and predicted tide height, 27-31 March 1984

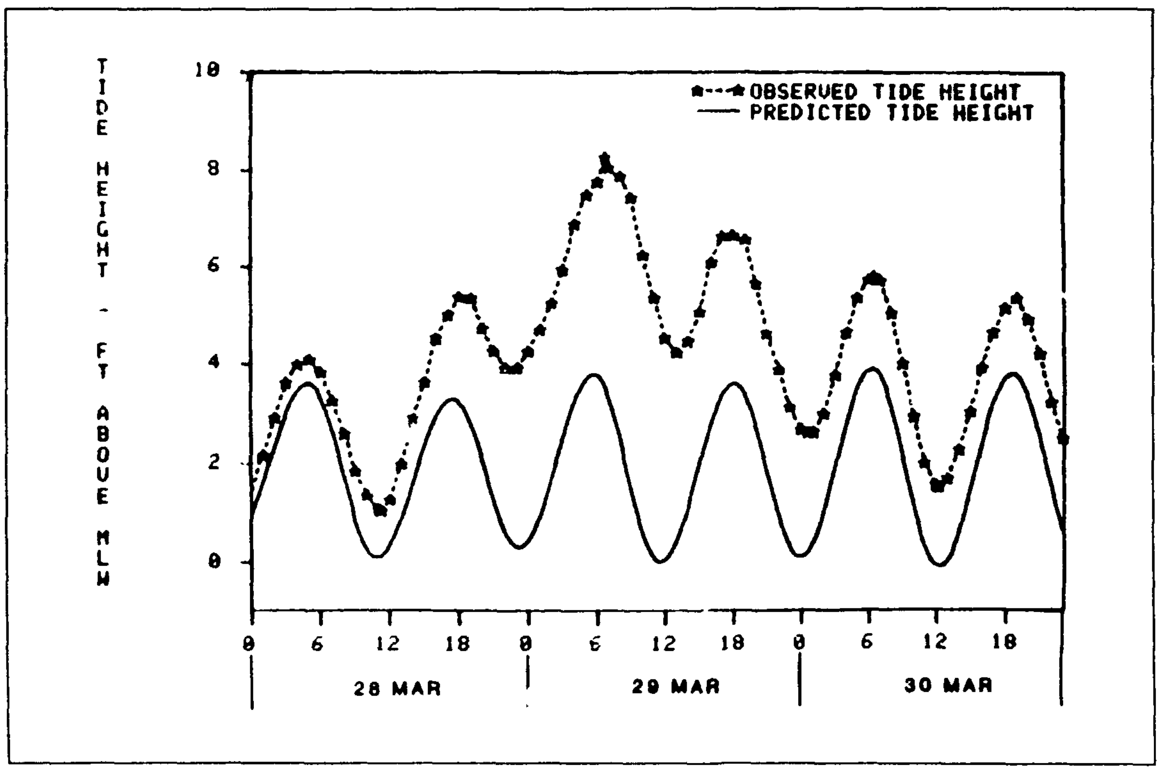


Figure 31. Observed and predicted tide height, 28-30 March 1984

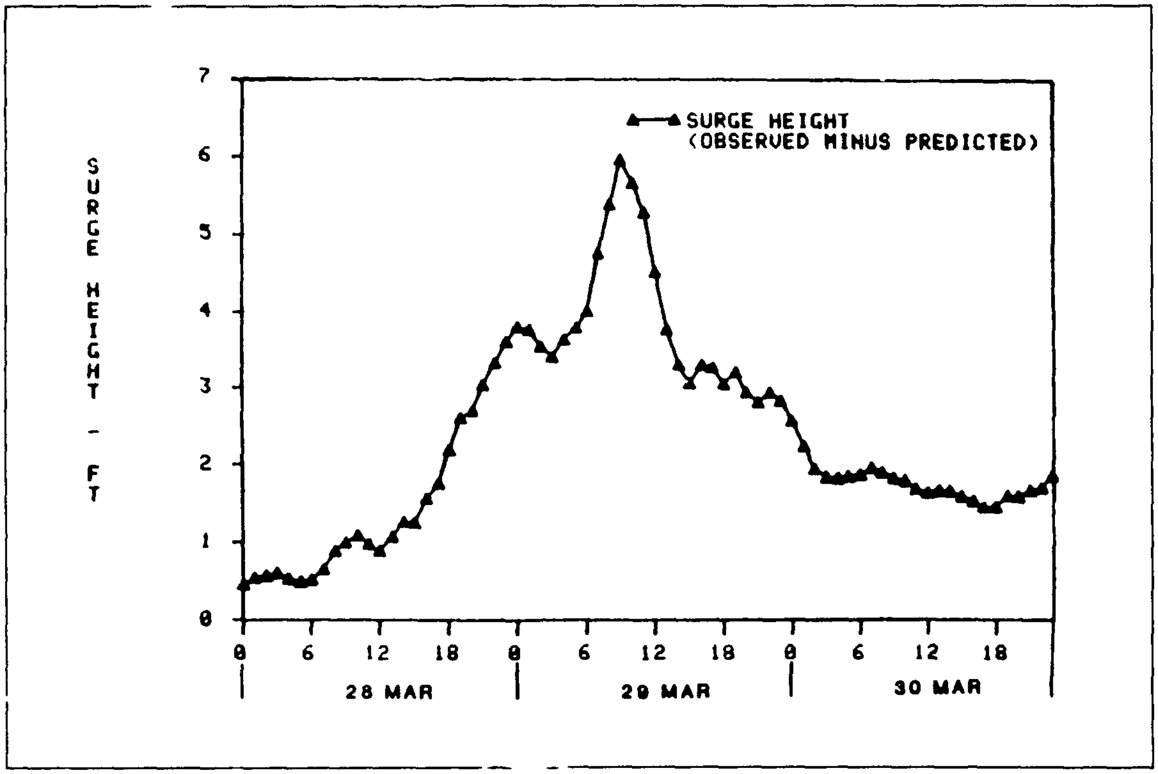


Figure 32. Surge height, 28-30 March 1984

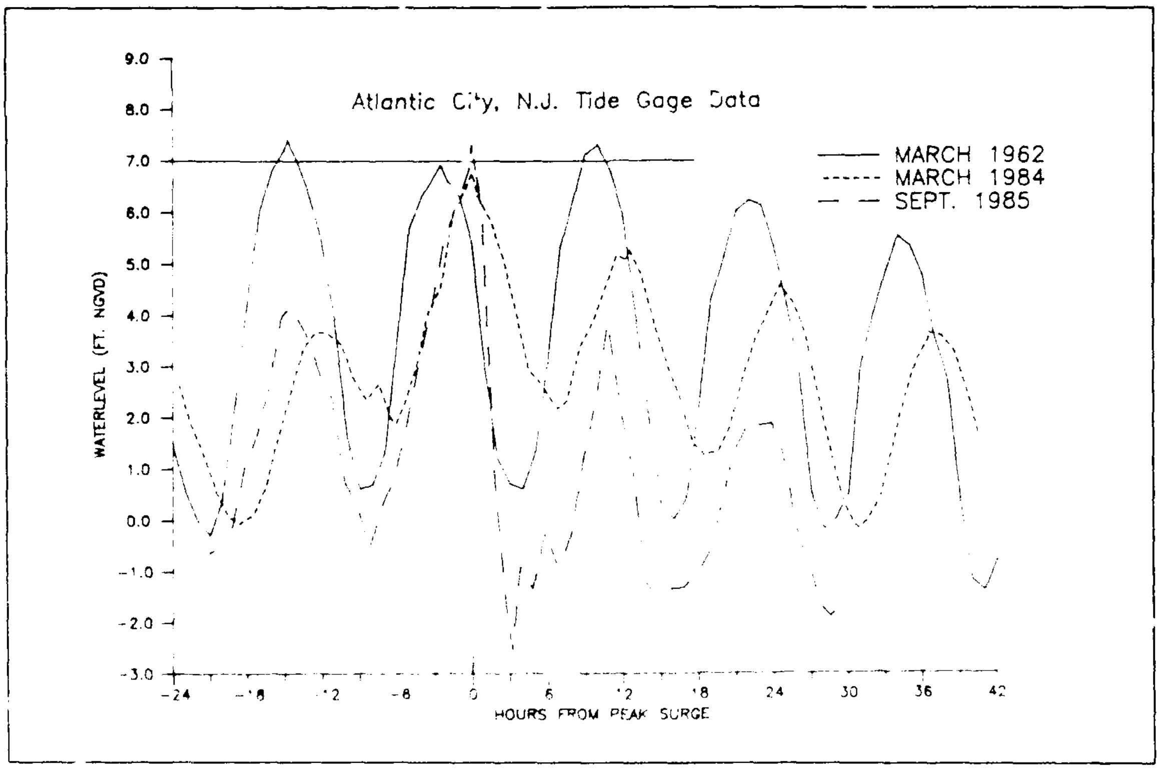


Figure 33. Comparison of water levels, 1962, 1984, and 1985 storms

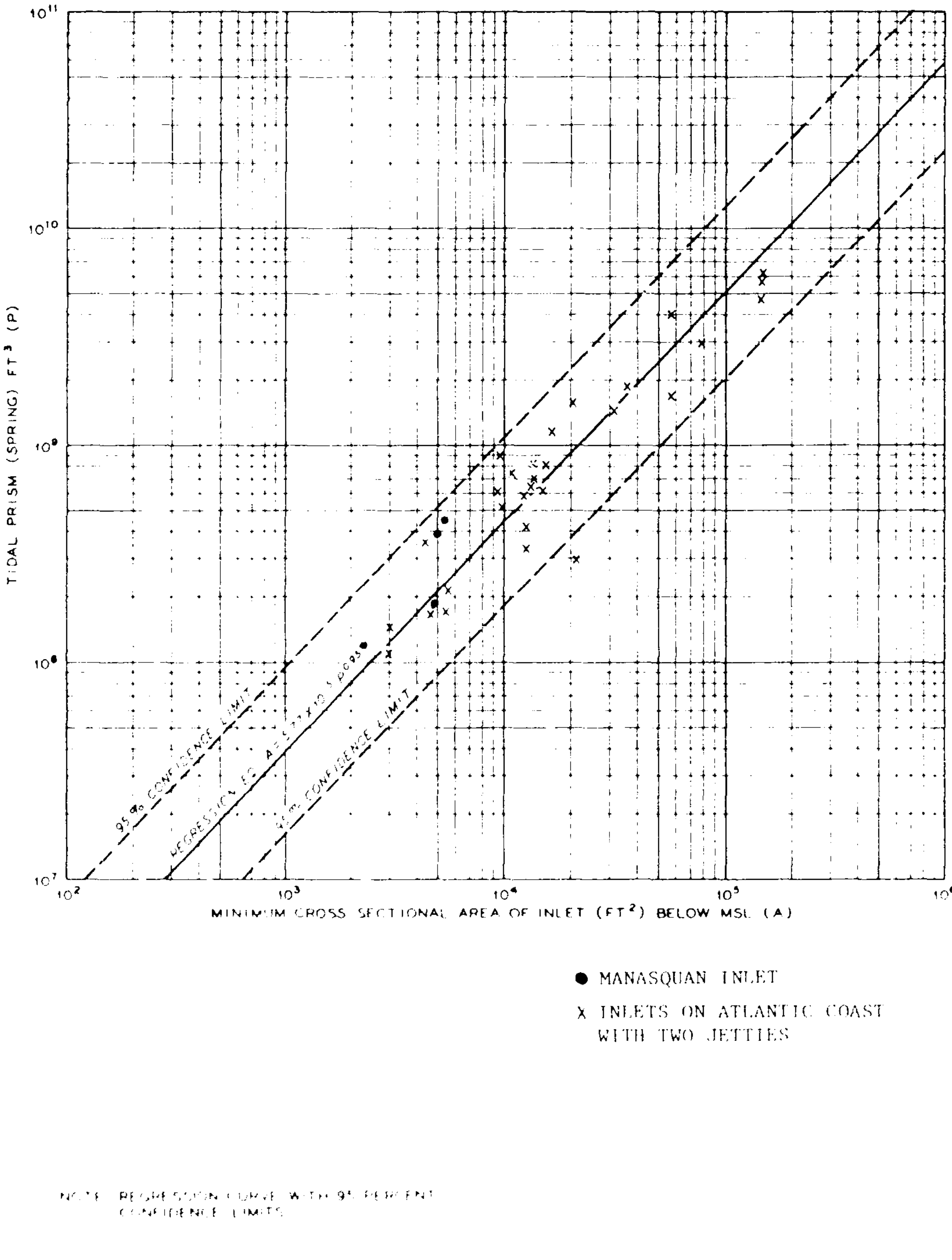


Figure 34. Tidal prism versus cross-sectional area

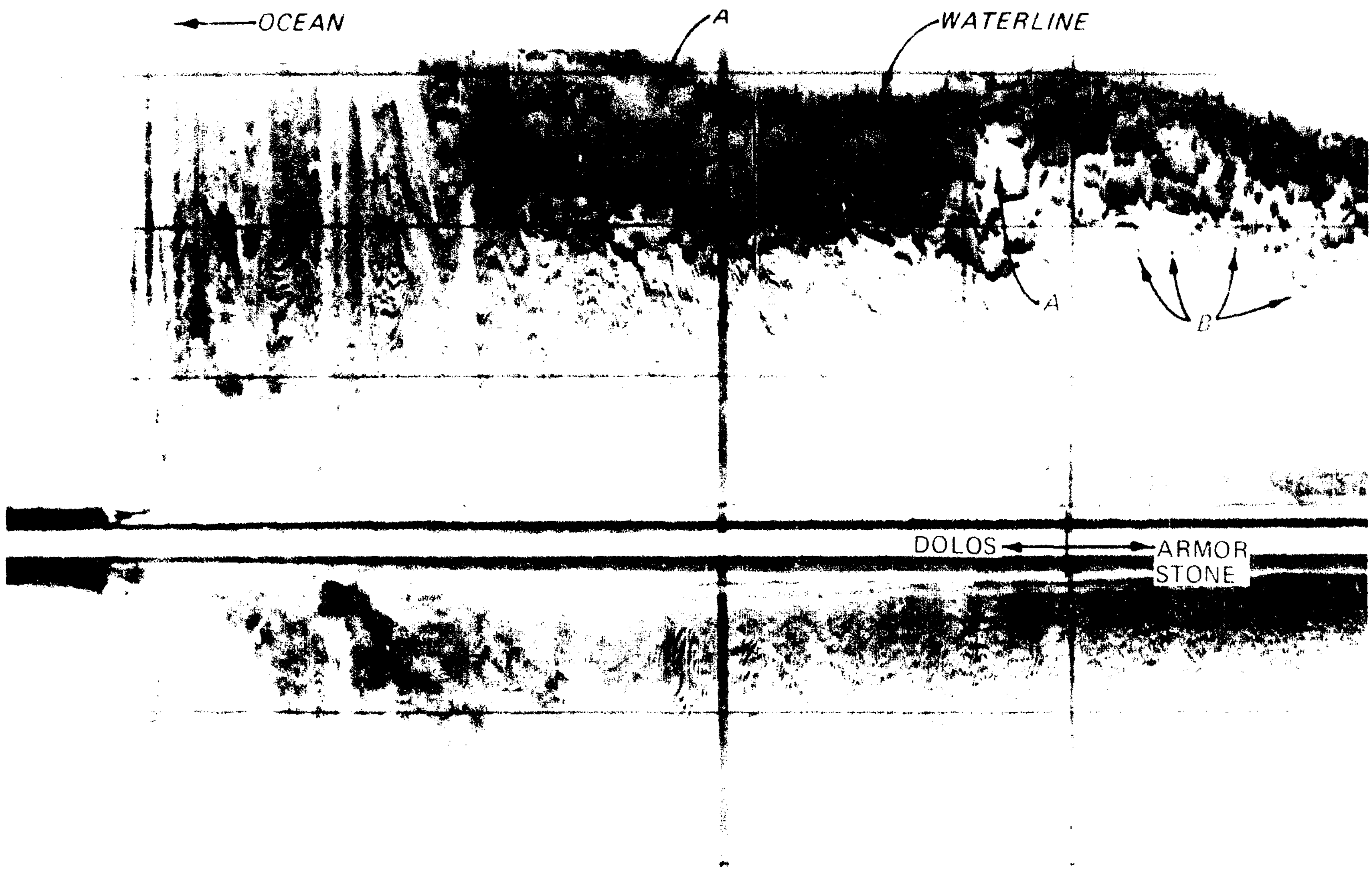


Figure 35 Sample side-scan sonar

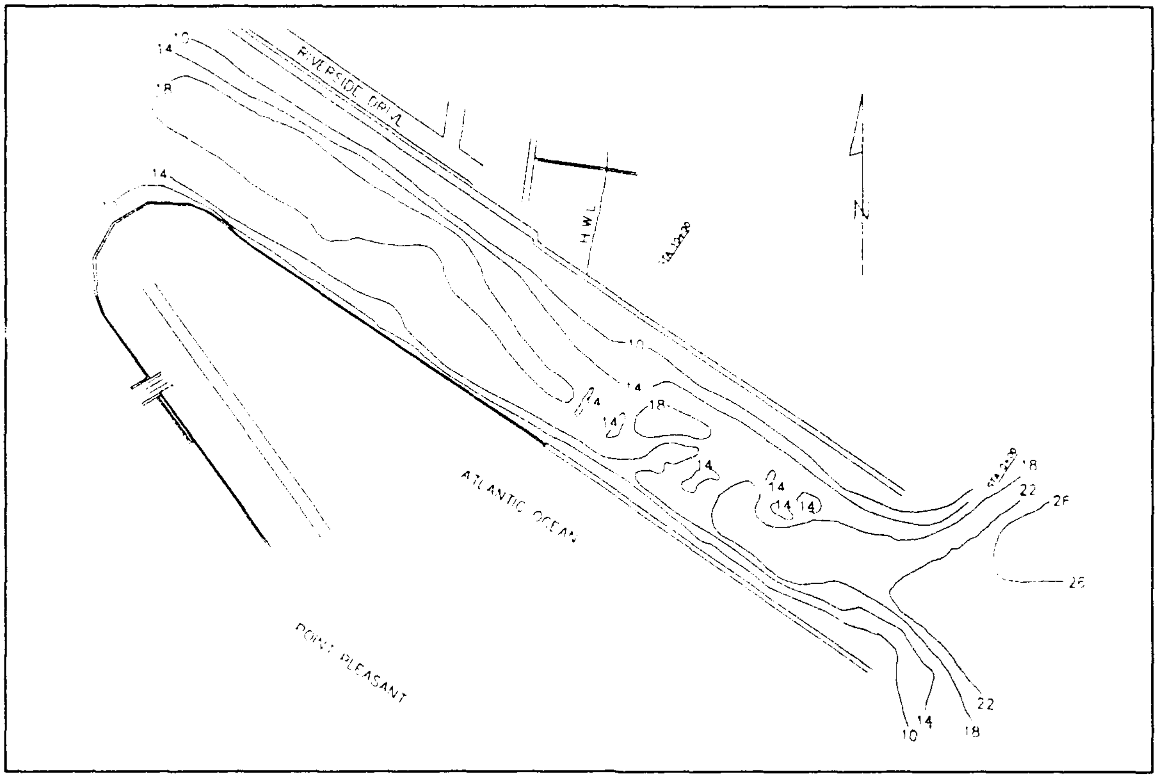


Figure 36. Inlet bathymetry, April 1977

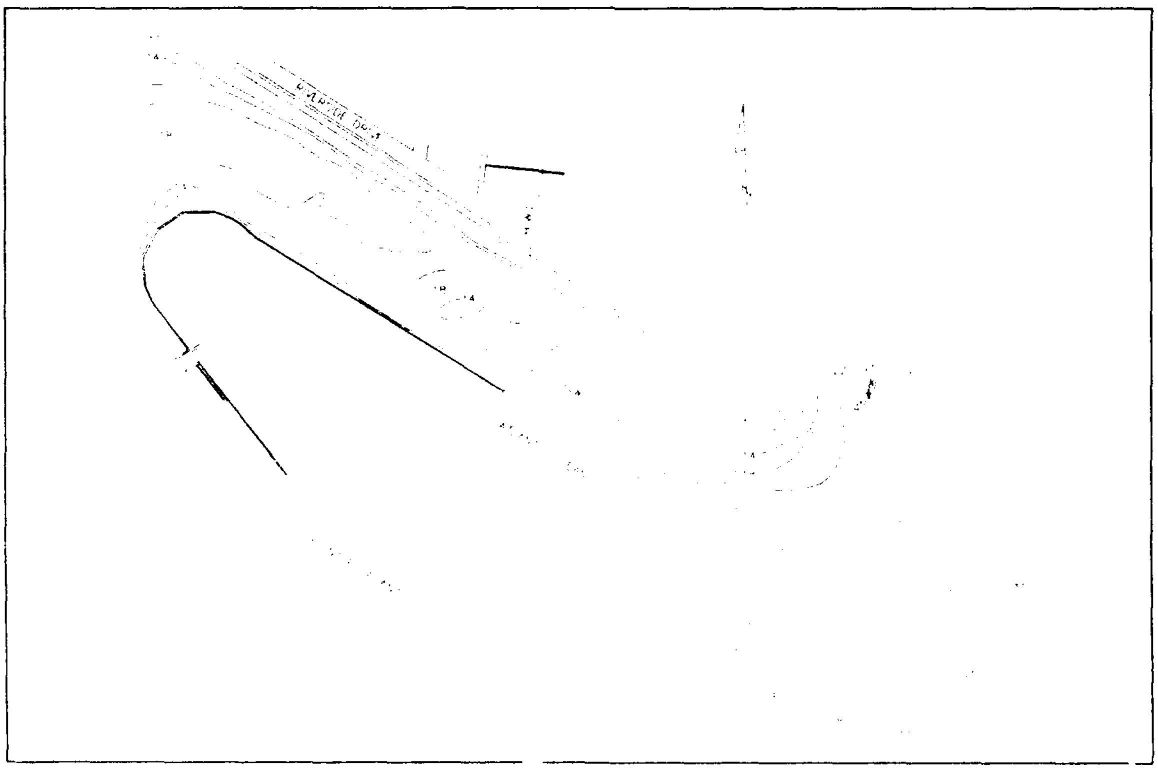


Figure 37. Inlet bathymetry, April 1984

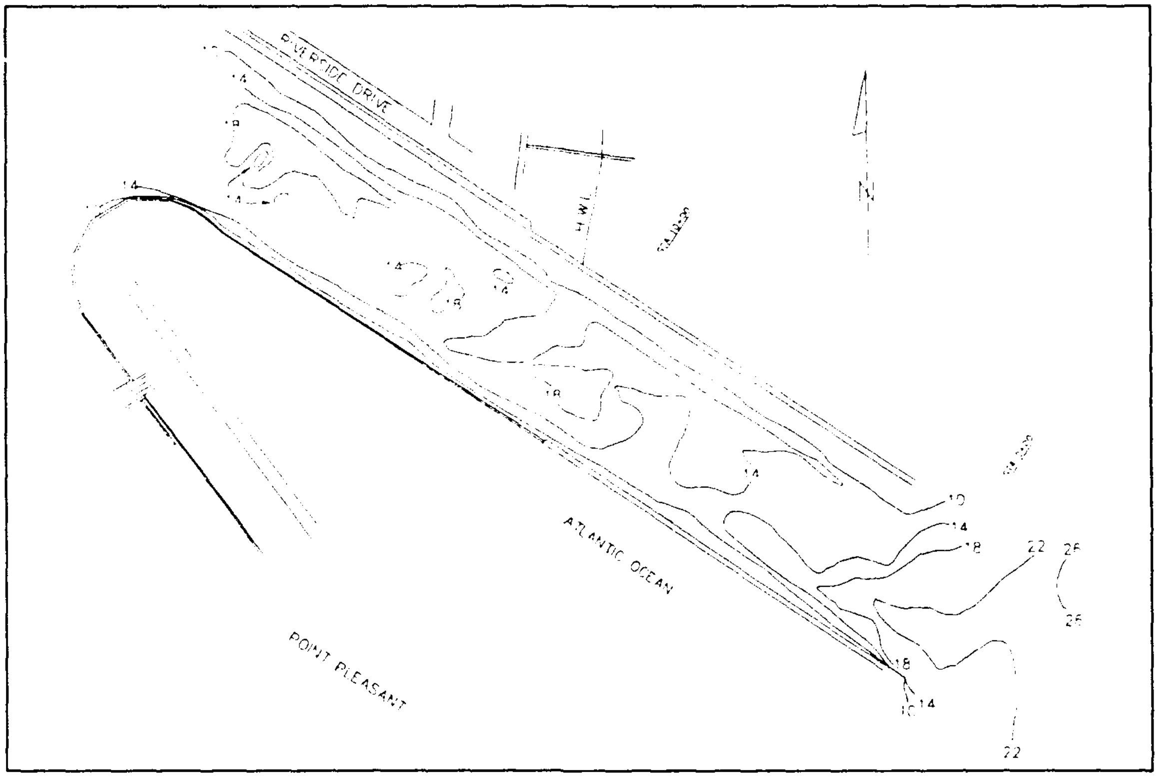


Figure 38. Inlet bathymetry, September 1984

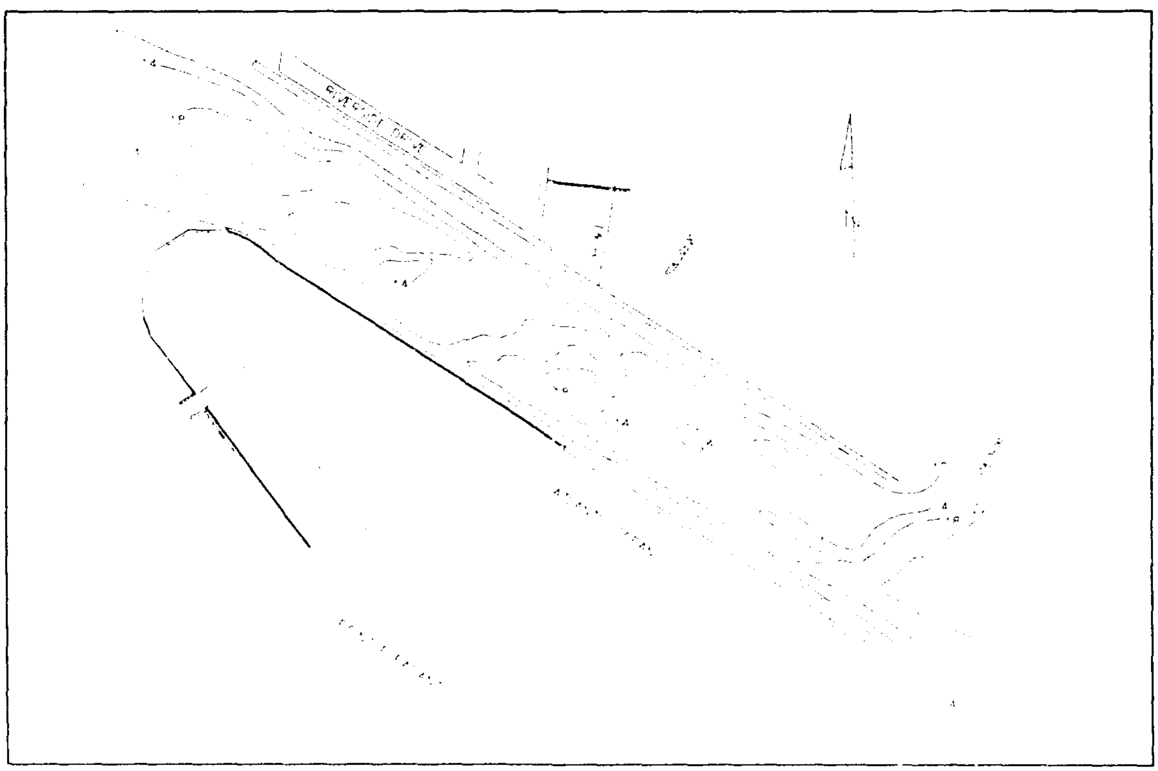


Figure 39. Inlet bathymetry, January 1986

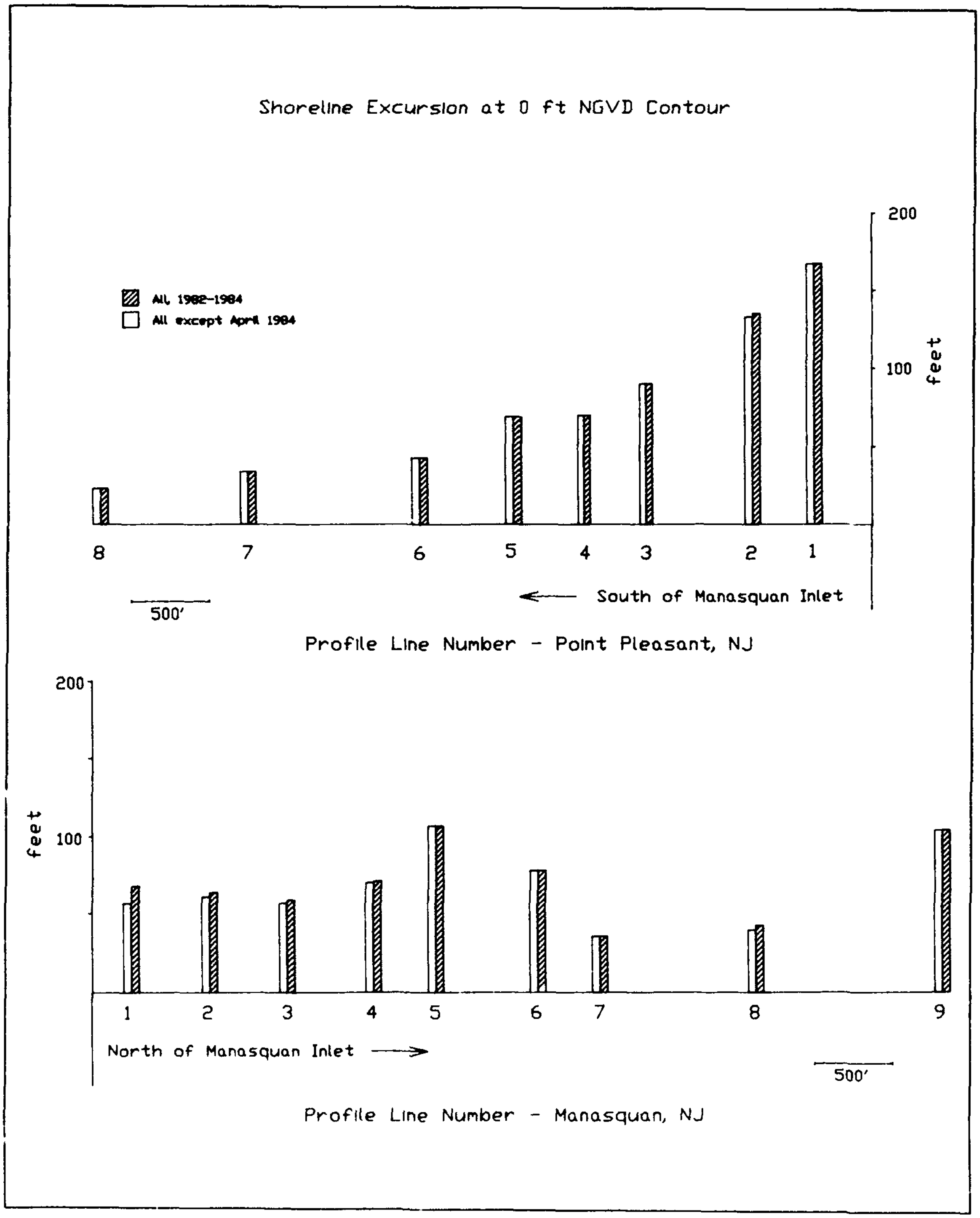


Figure 40. Profile variability for each profile line

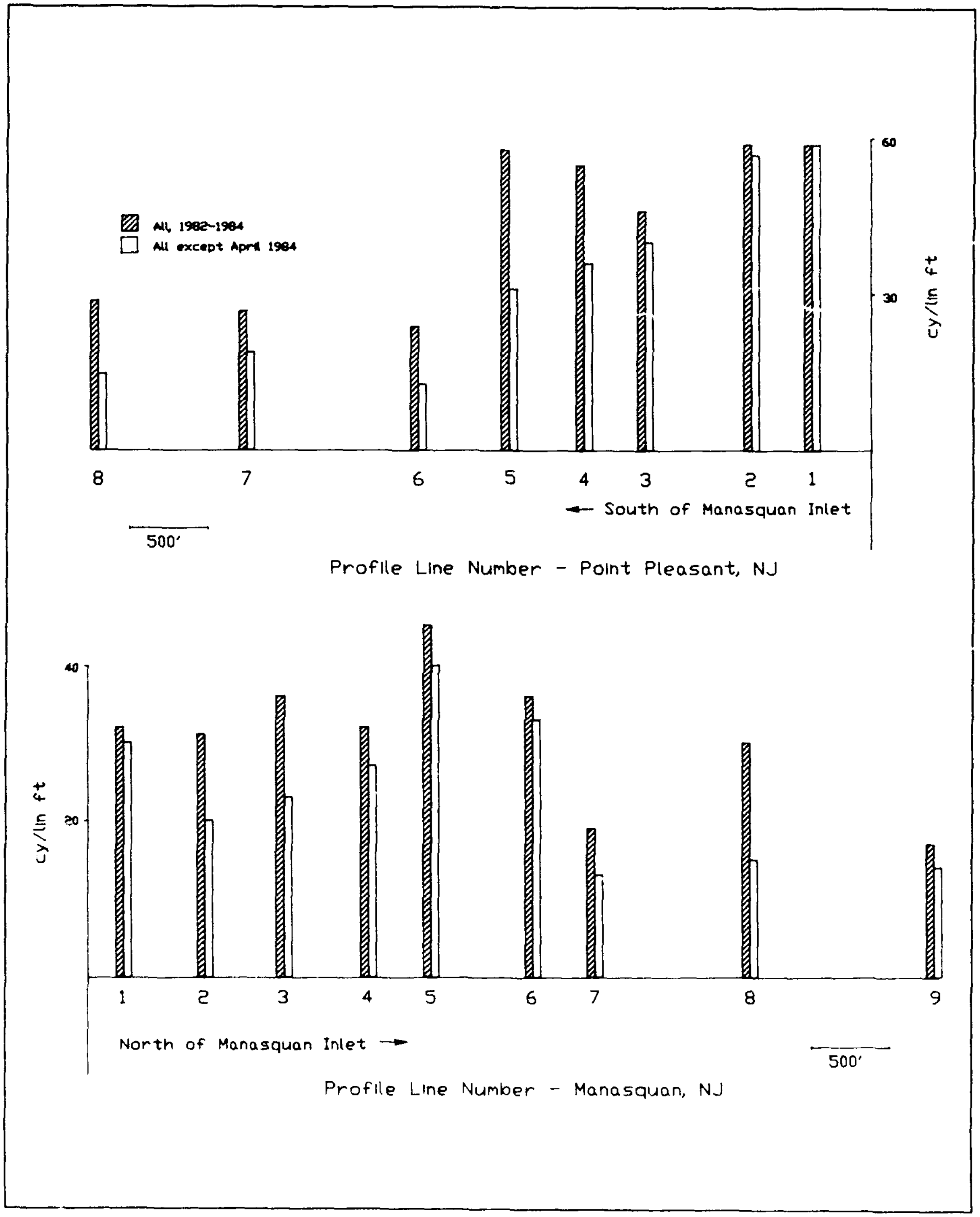


Figure 41. Profile volume envelope for each profile line

Δz (feet)	SOUTH JETTY April 1982 to September 1983	NORTH JETTY March 1983 to September 1983
$\Delta z = 0$	3	18
$0 < \Delta z \leq 0.1$	22	32
$0.1 < \Delta z \leq 0.2$	1	3
$0.2 < \Delta z \leq 0.3$	1	3
$\Delta z > 0.3$	2	1
(Total upward)	(26)	(39)
$-0.1 \leq \Delta z < 0$	36	35
$-0.2 \leq \Delta z < -0.1$	13	6
$-0.3 \leq \Delta z < -0.2$	11	0
$\Delta z < -0.3$	11	2
(Total downward)	(71)	(43)

Note: positive Δz is upward movement, negative is downward.

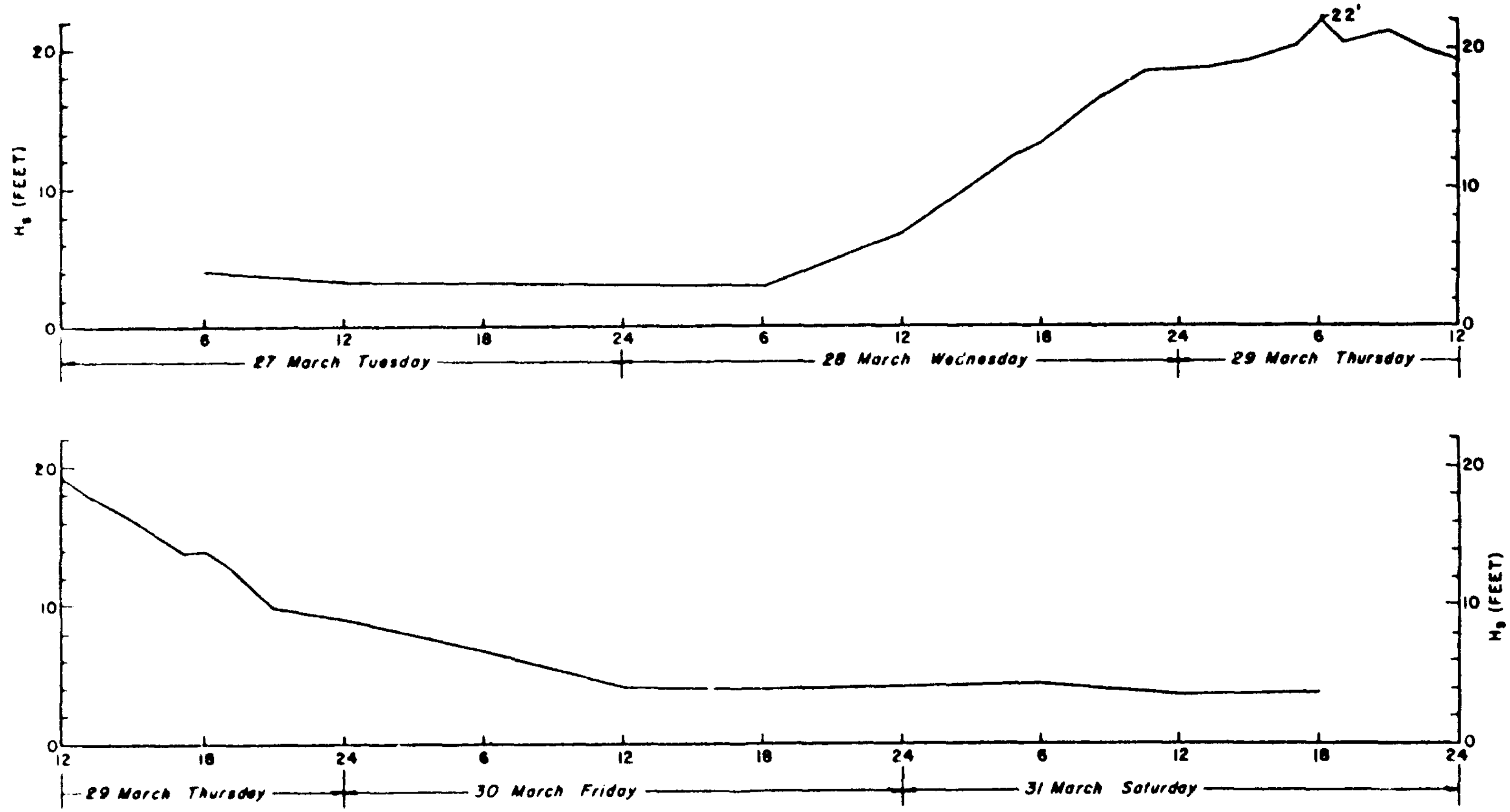
Figure 42. Dolos elevation change determined by leveling; percent of change

Δz (feet)	SOUTH JETTY	NORTH JETTY
$\Delta z = 0$	3	4
$0 < \Delta z \leq 0.1$	3	23
$0.1 < \Delta z \leq 0.2$	0	7
$0.2 < \Delta z \leq 0.3$	1	0
$\Delta z > 0.3$	2	0
(Total upward)	(6)	(30)
$-0.1 \leq \Delta z < 0$	24	26
$-0.2 \leq \Delta z < -0.1$	22	10
$-0.3 \leq \Delta z < -0.2$	14	10
$\Delta z < -0.3$	31	20
(Total downward)	(91)	(66)

Figure 43. Dolos elevation changes determined by photogammetry, March/April

Figure 44. Broken dolosse with broken reinforcing steel

Figure 45. Cracked dolosse



Note
"SIGNIFICANT WAVE HEIGHT" is average height of highest one-third of the waves over a given period.

SAMPLING DURATION = 20 minutes

SAMPLING INTERVAL = 6 hrs ordinarily, 1 to 2 hours when waves exceed 3 meters
(each point represents 20 minutes of data)

MANASQUAN, NJ WAVE GAGE
27 - 31 MARCH 1984

Figure 46. Measured significant wave heights, 27-31 March 1984

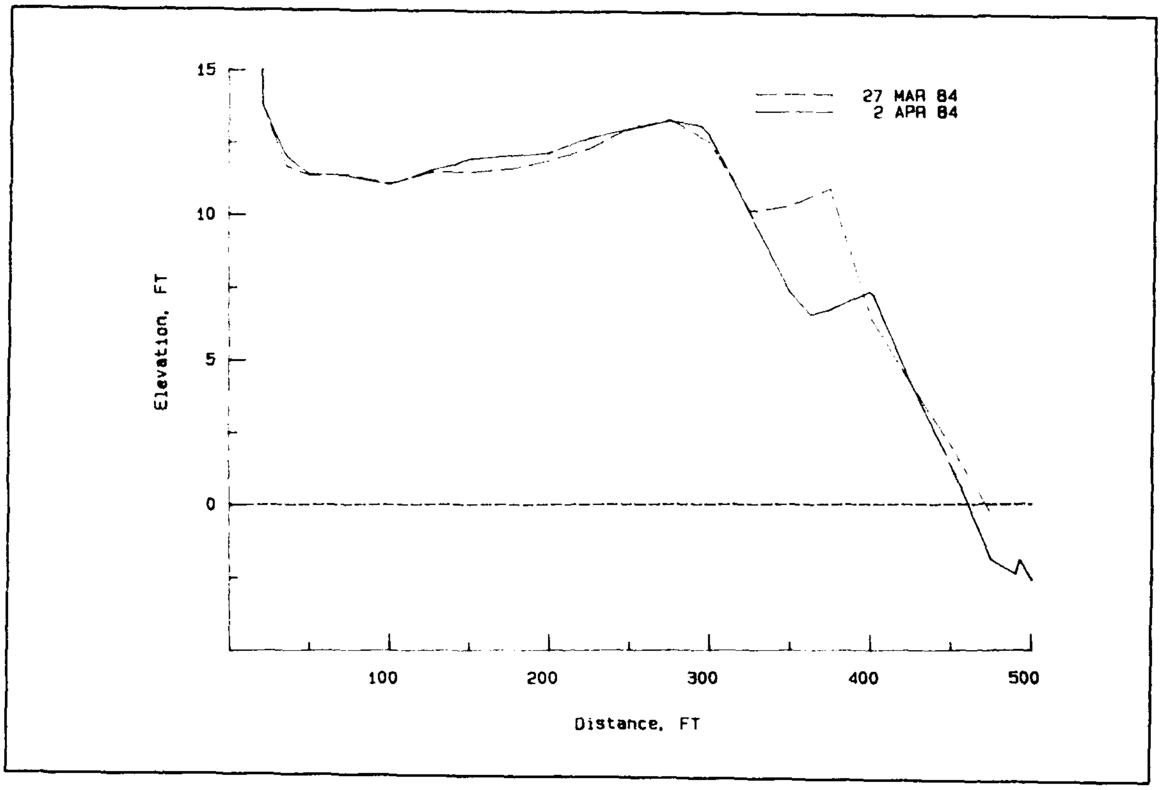


Figure 47. Profile changes, 27 March and 2 April 1984, profile line PP-1

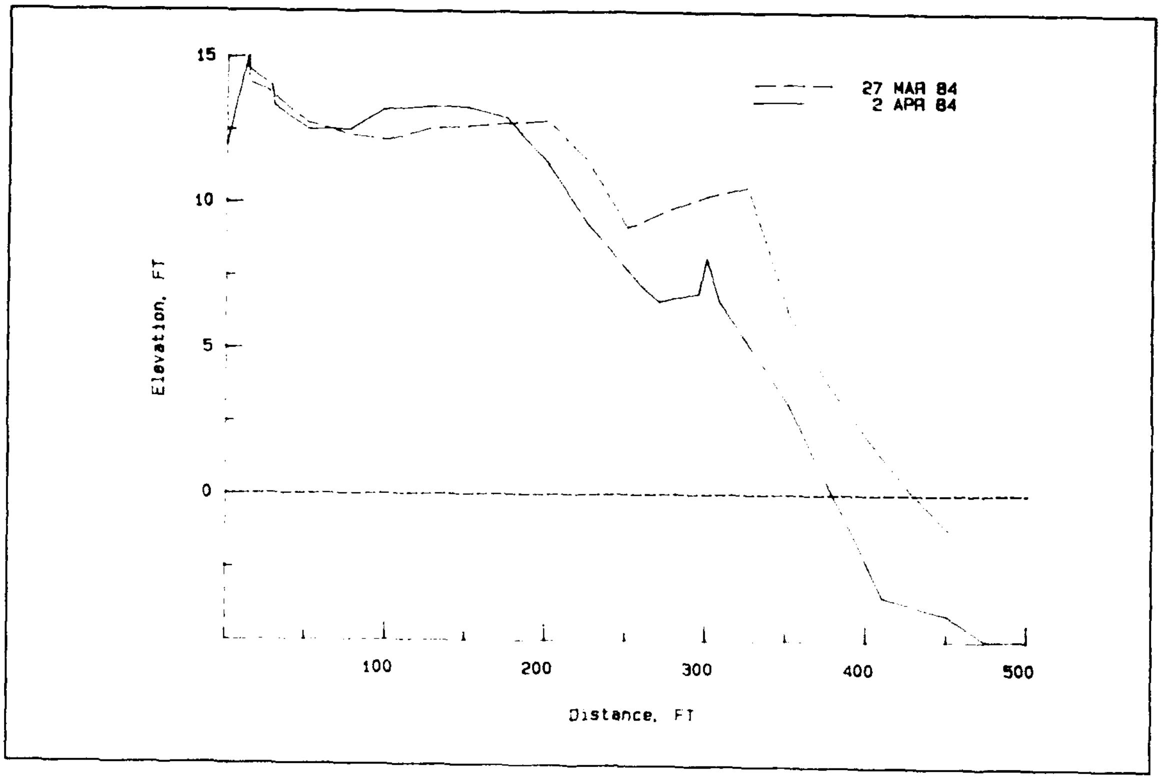


Figure 48. Profile changes, 27 March and 2 April 1984, profile line PP-2

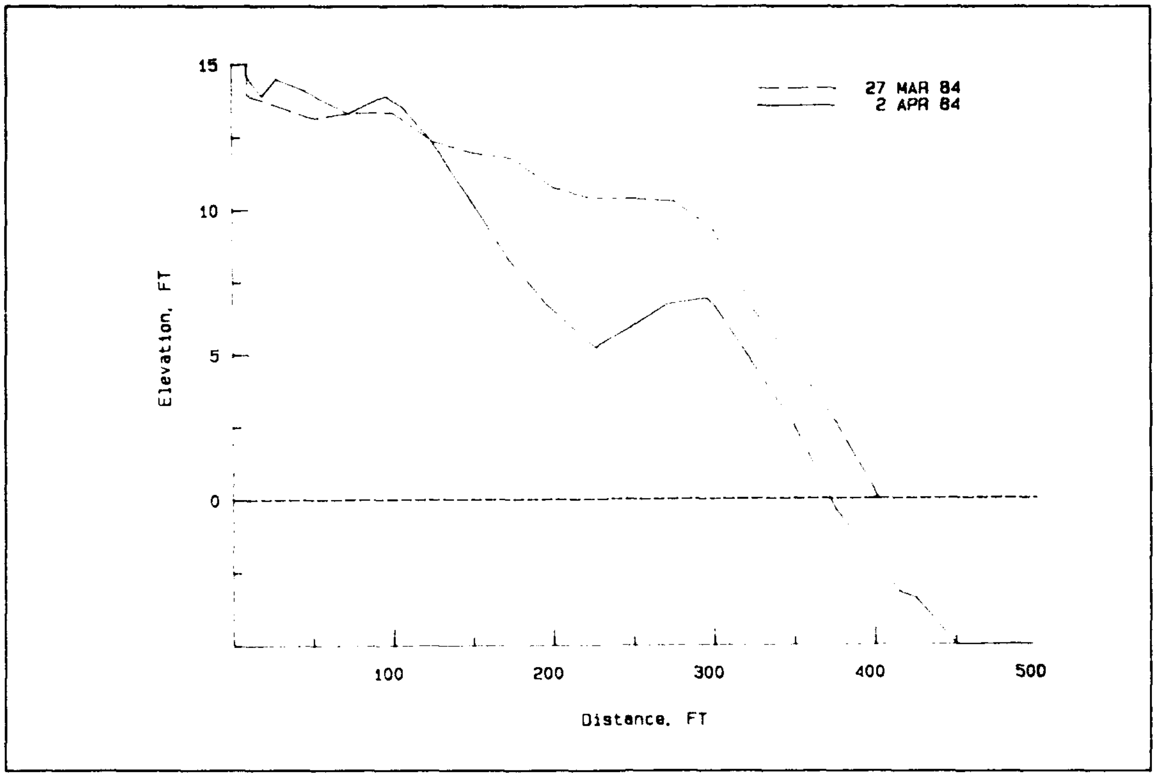


Figure 49. Profile changes, 27 March and 2 April 1984, profile line PP-3

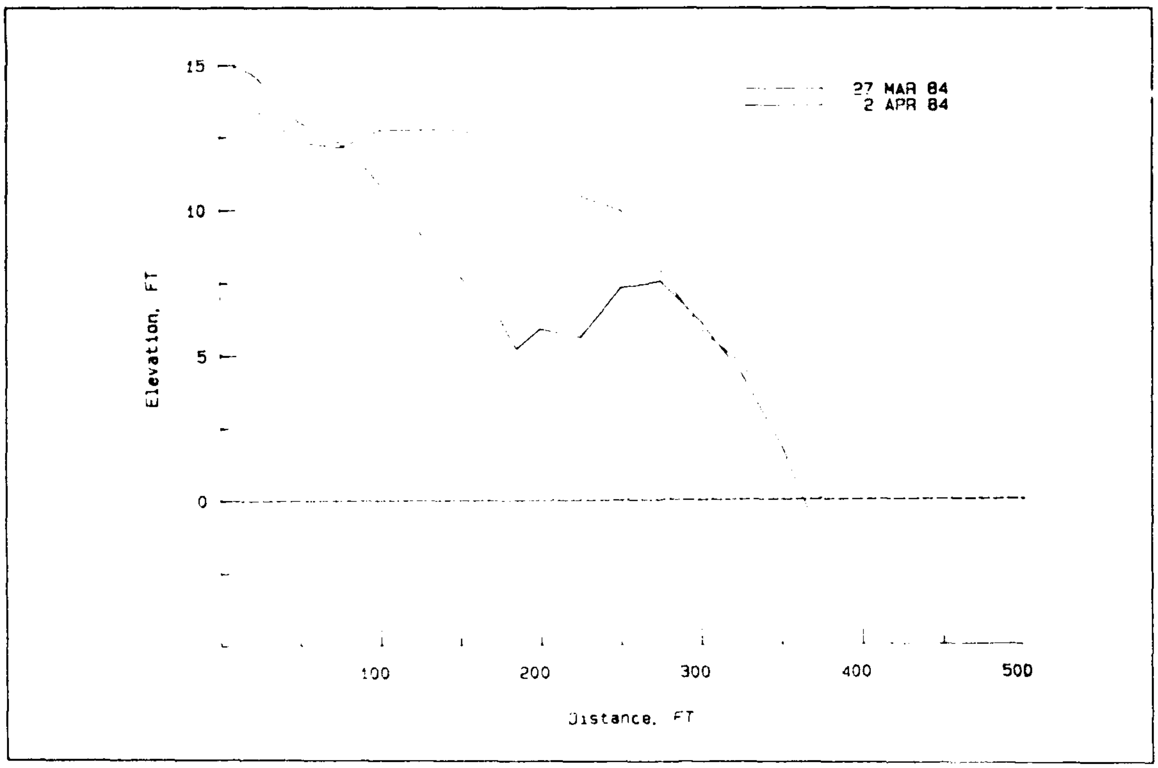


Figure 50. Profile changes, 27 March and 2 April 1984, profile line PP-4

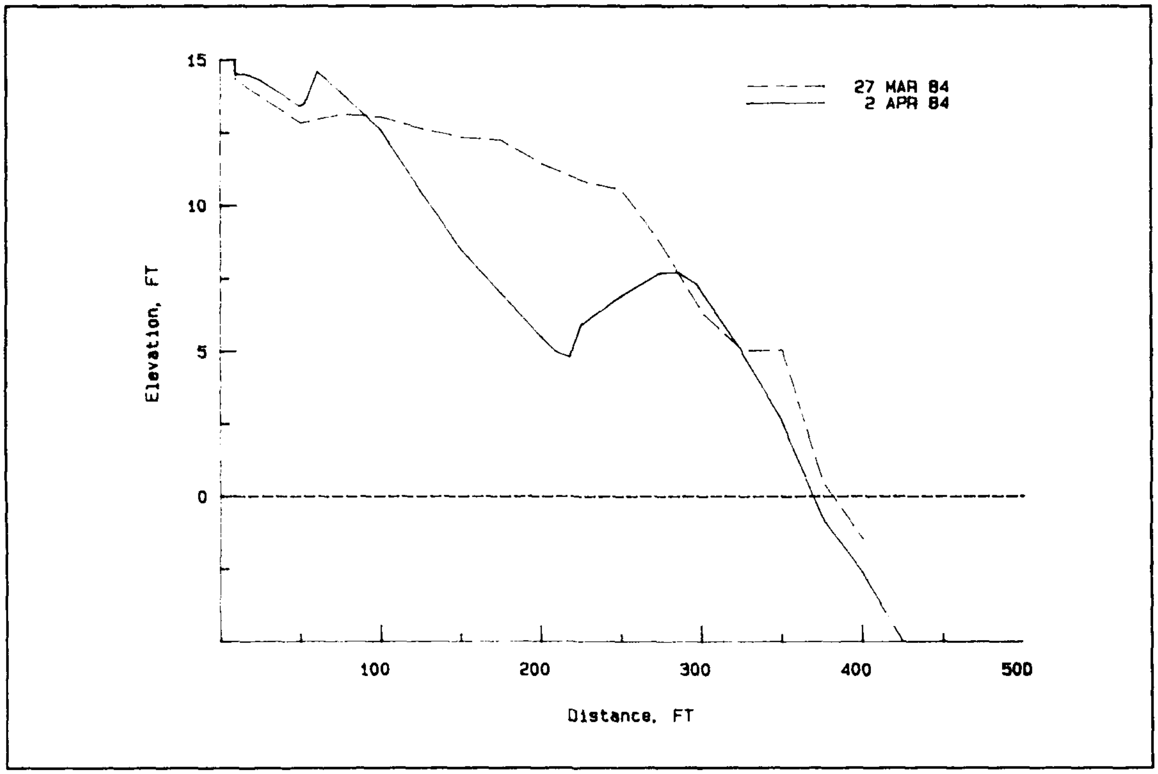


Figure 51. Profile changes, 27 March and 2 April 1984, profile line PP-5

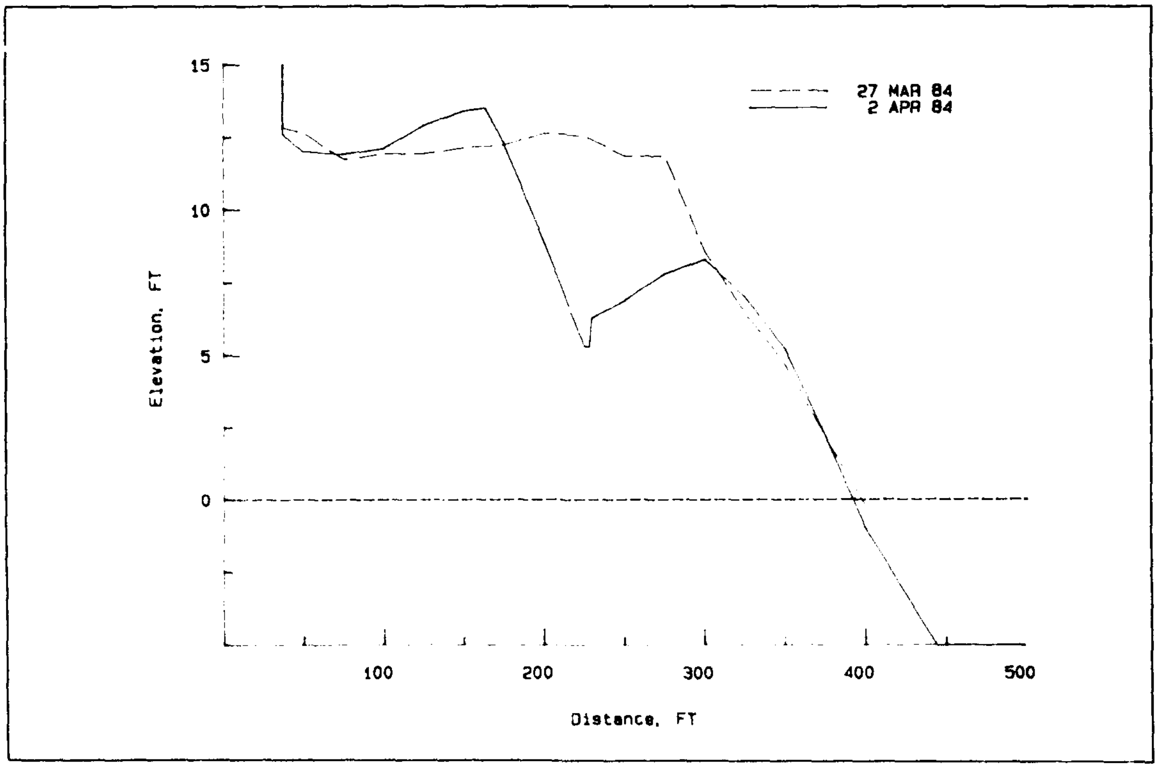


Figure 52. Profile changes, 27 March and 2 April 1984, profile line PP-6

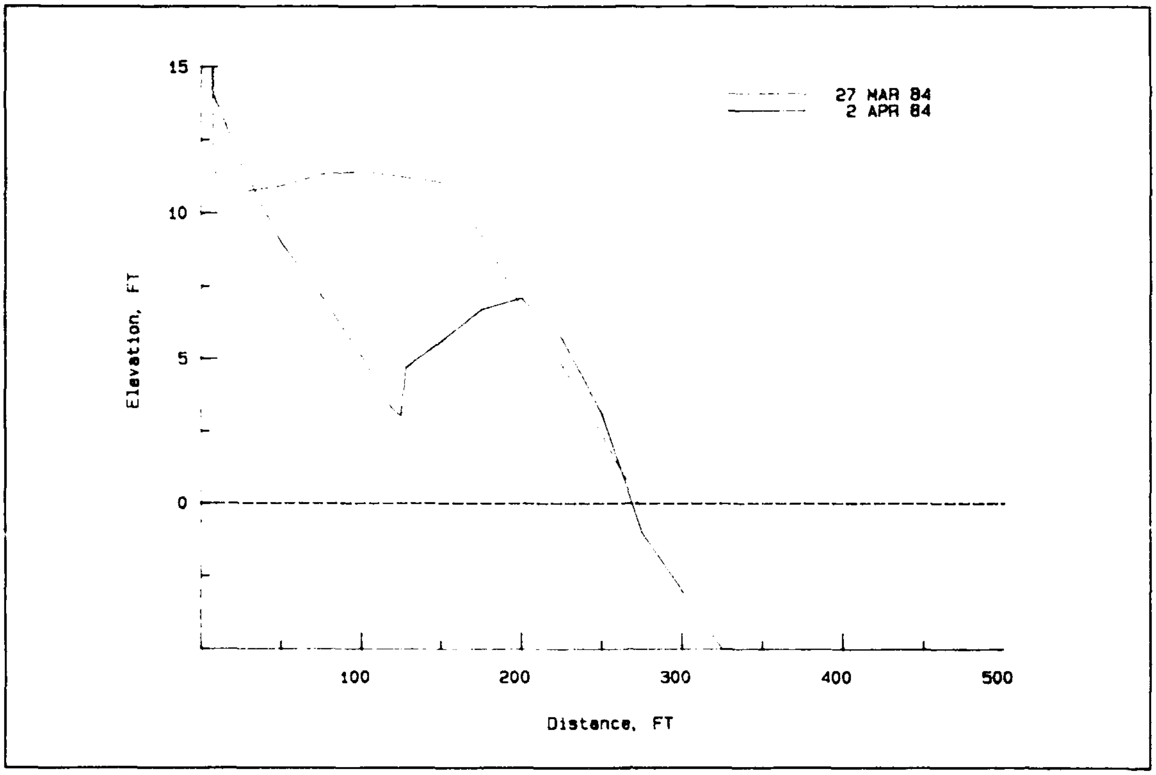


Figure 53. Profile changes, 27 March and 2 April 1984, profile line PP-7

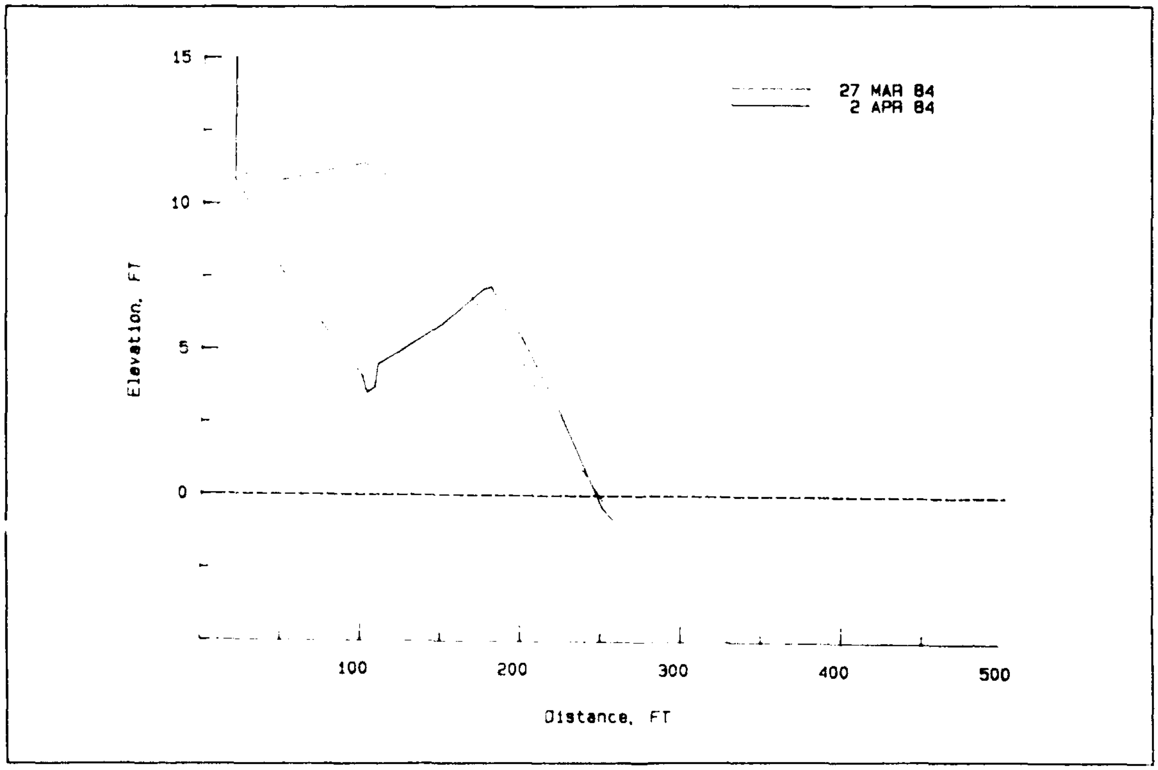


Figure 54. Profile changes, 27 March and 2 April 1984, profile line PP-8

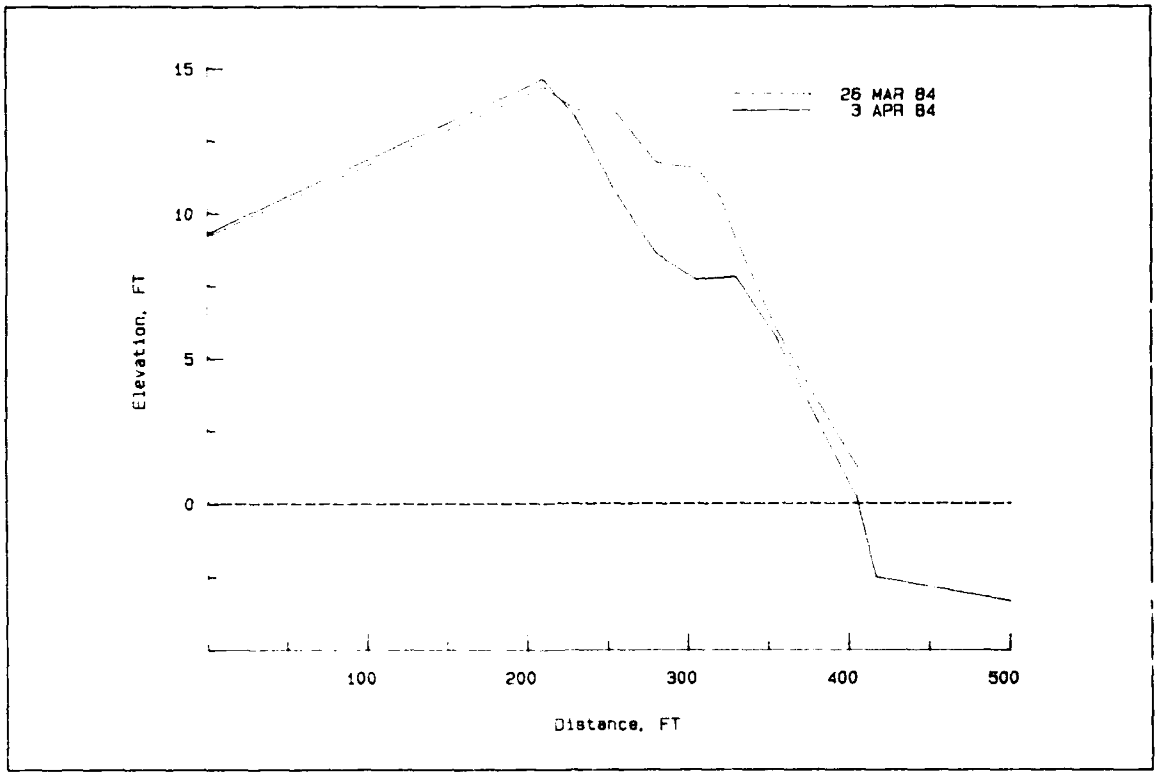


Figure 55. Profile changes, 27 March and 2 April 1984, profile line M-1

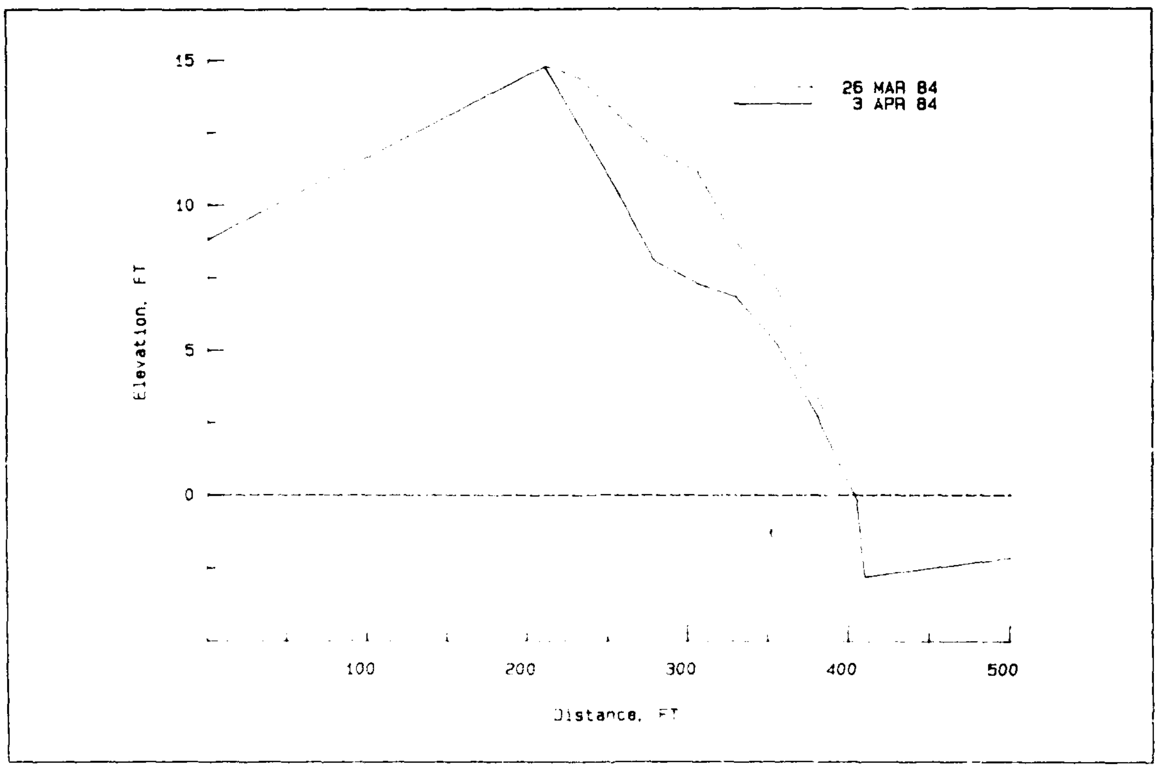


Figure 56. Profile changes, 27 March and 2 April 1984, profile line M-2

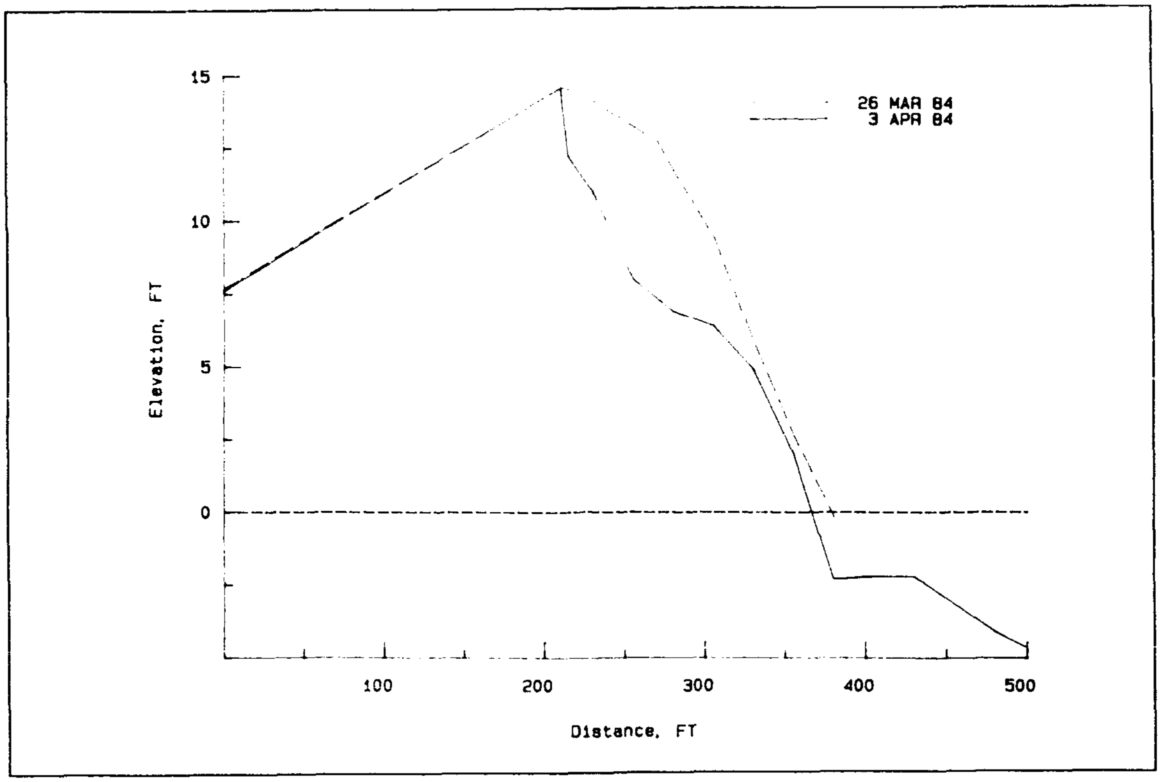


Figure 57. Profile changes, 27 March and 2 April 1984, profile line M-3

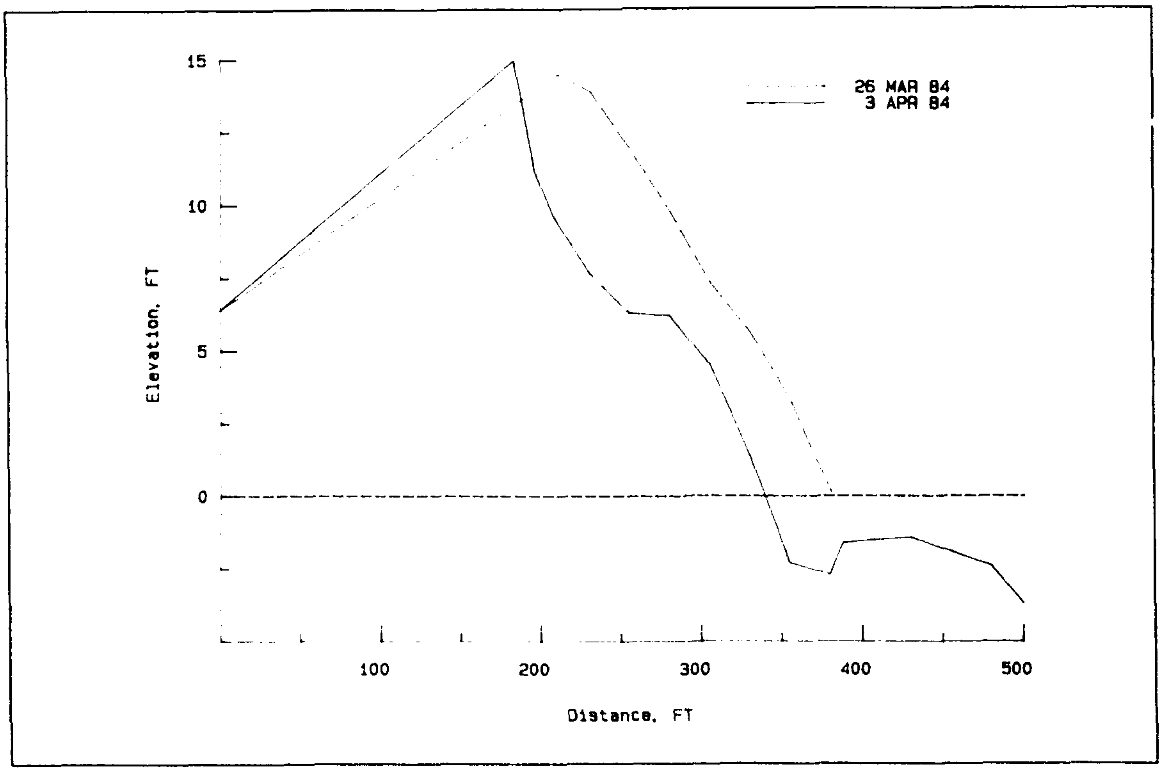


Figure 58. Profile changes, 27 March and 2 April 1984, profile line M-4

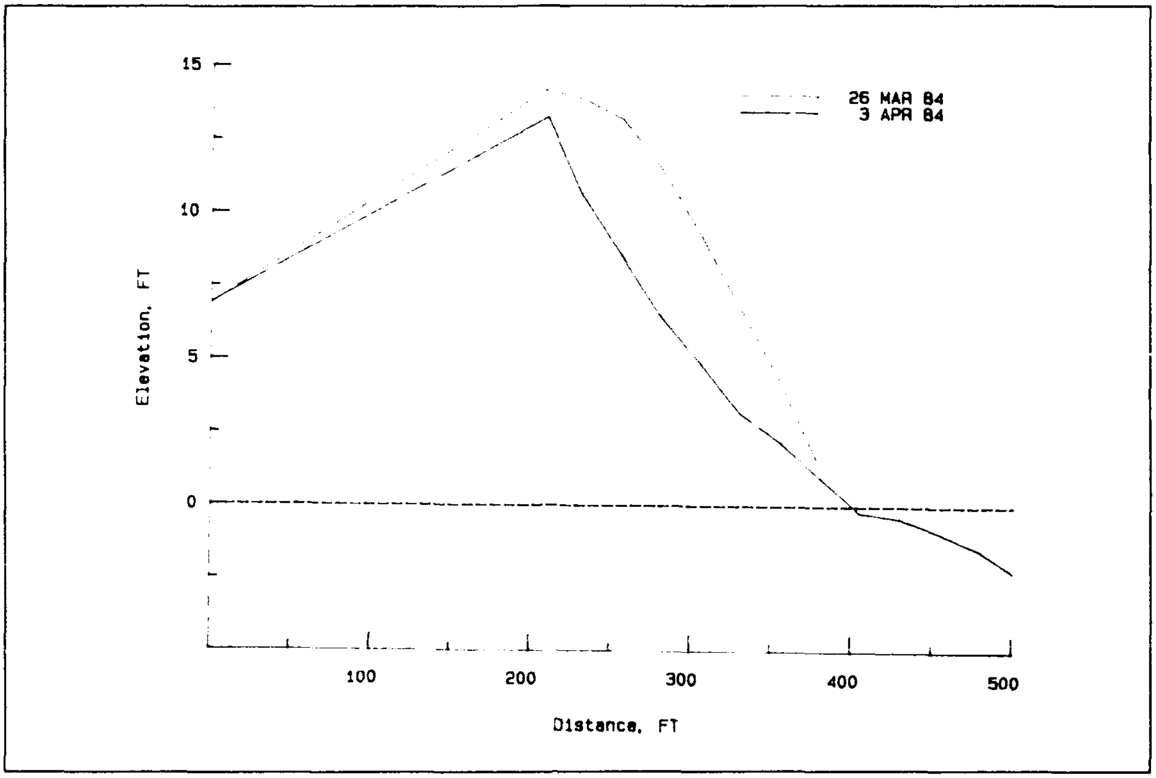


Figure 59. Profile changes, 27 March and 2 April 1984, profile line M-5

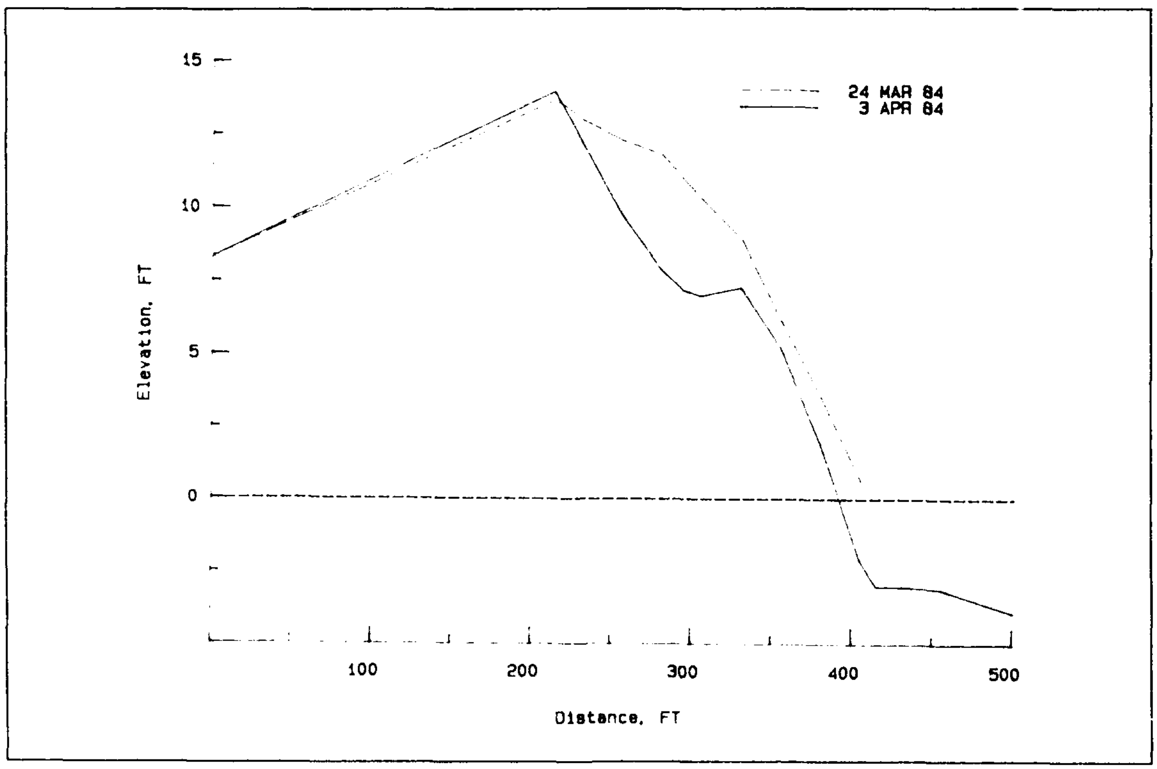


Figure 60. Profile changes, 27 March and 2 April 1984, profile line M-6

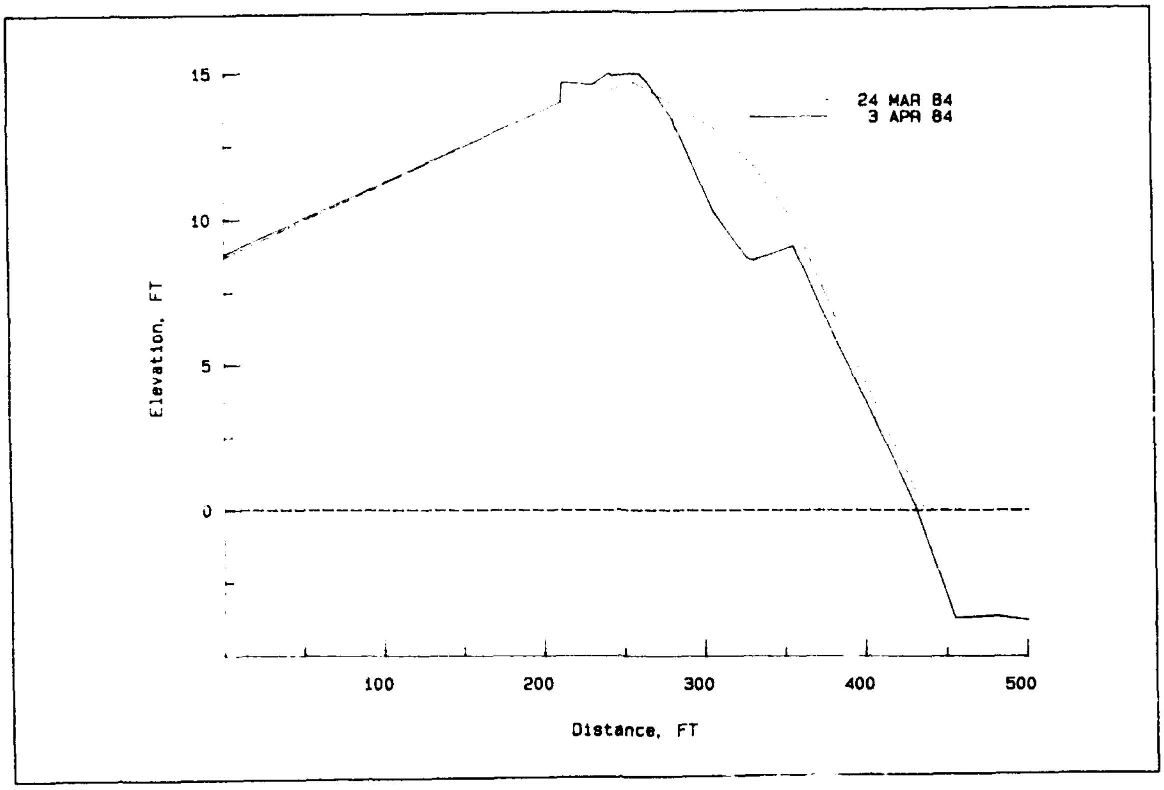


Figure 61. Profile changes, 27 March and 2 April 1984, profile line M-7

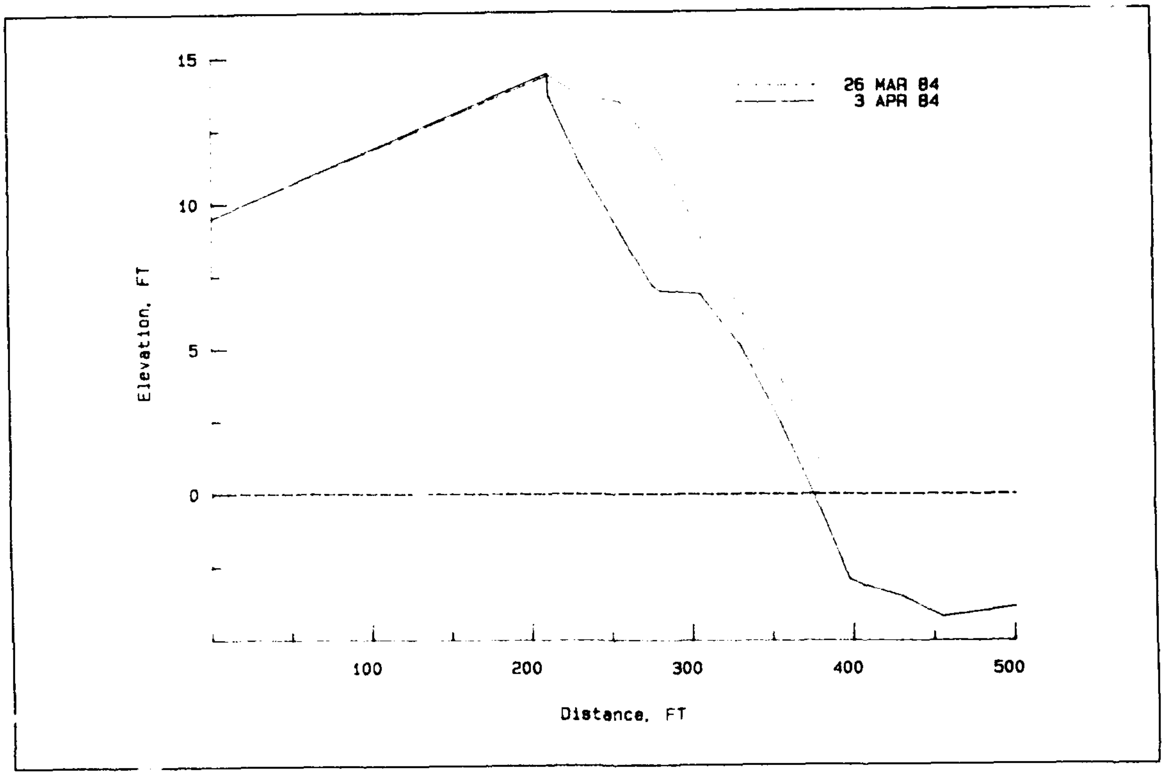


Figure 62. Profile changes, 27 March and 2 April 1984, profile line M-8

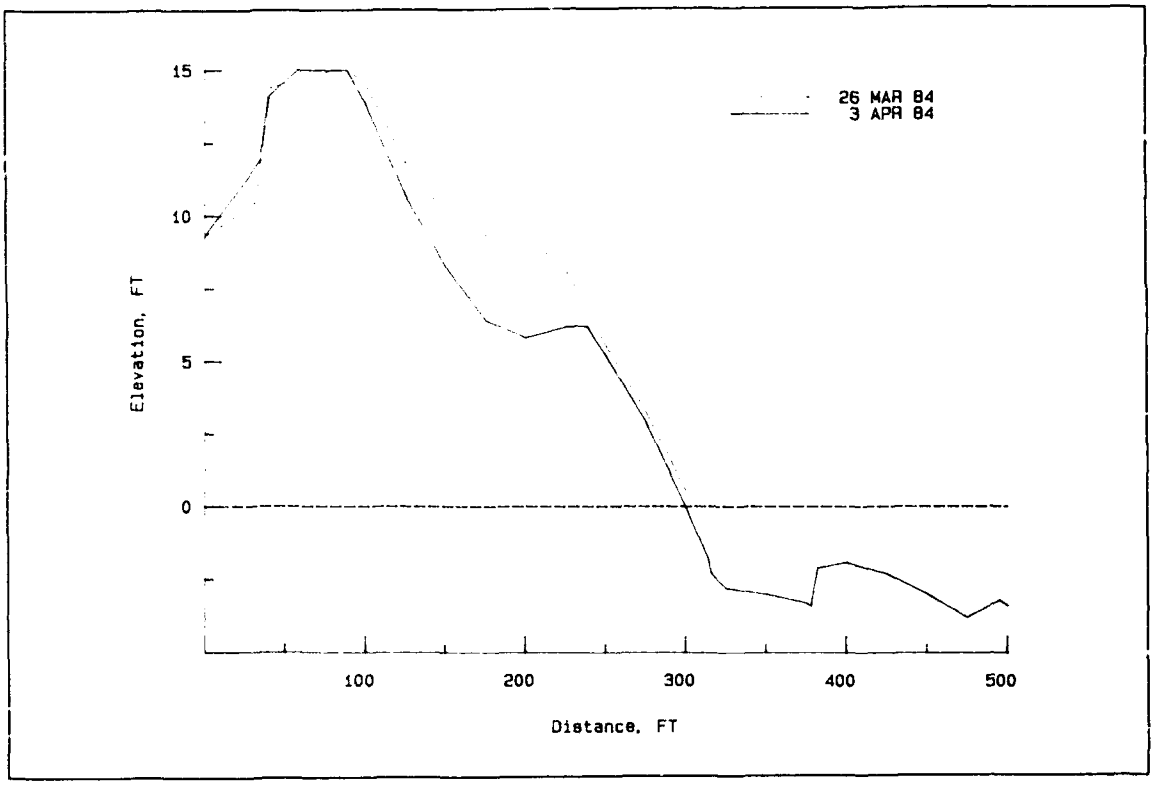


Figure 63. Profile changes, 27 March and 2 April 1984, profile line M-9

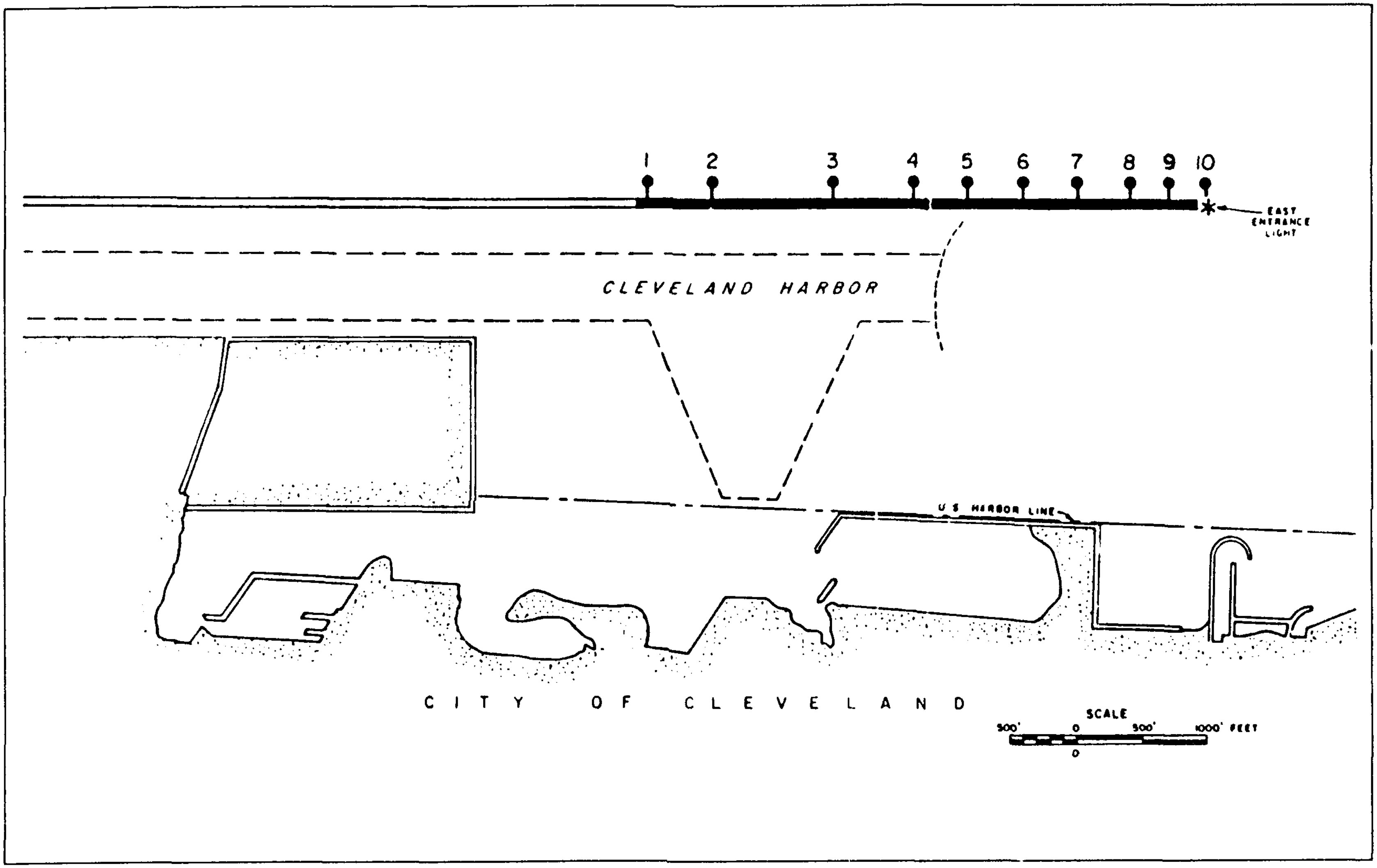


Figure 64. Cleveland Harbor, Ohio, monitoring program station locations

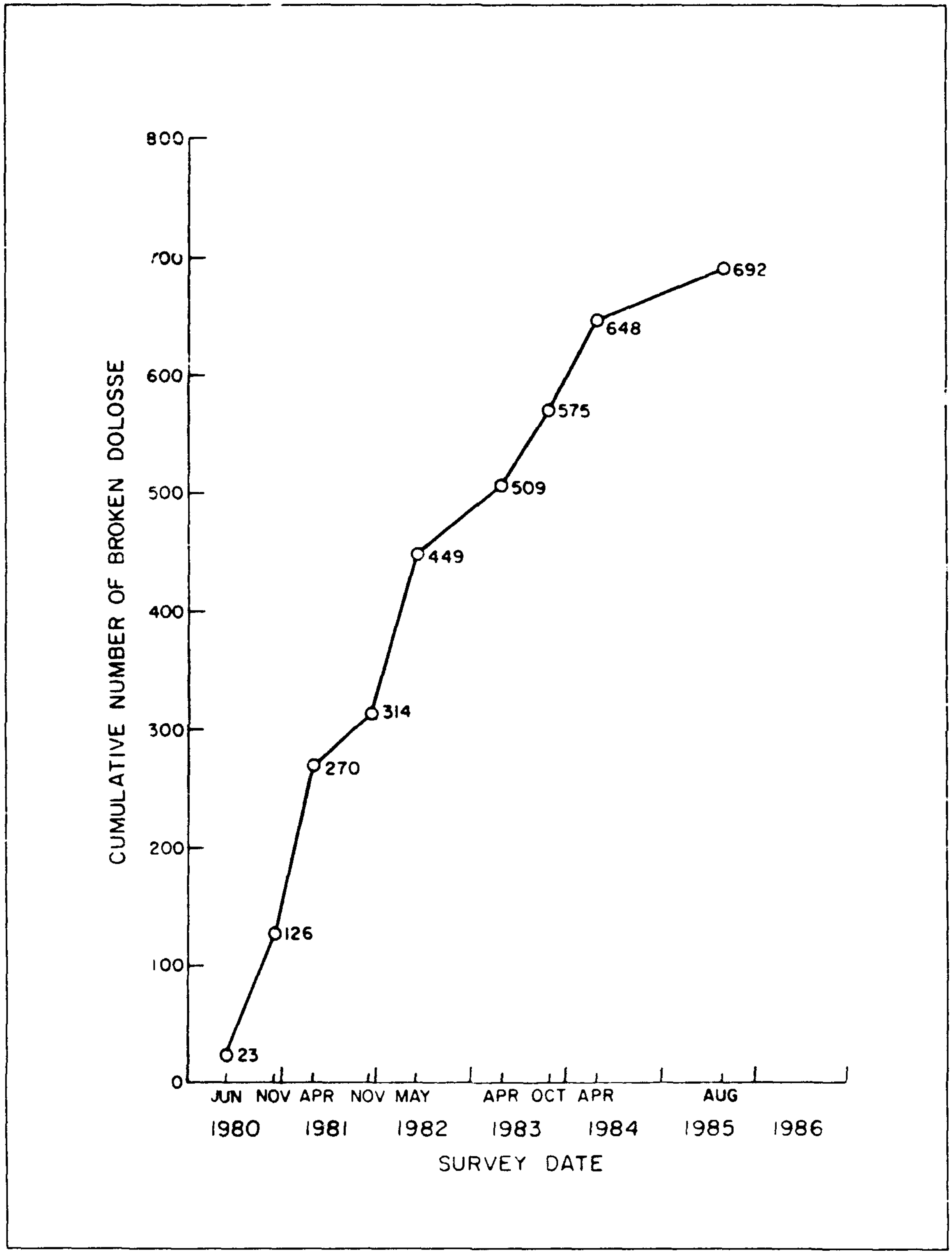


Figure 65. Cumulative number of broken dolosse, Cleveland Harbor East Breakwater

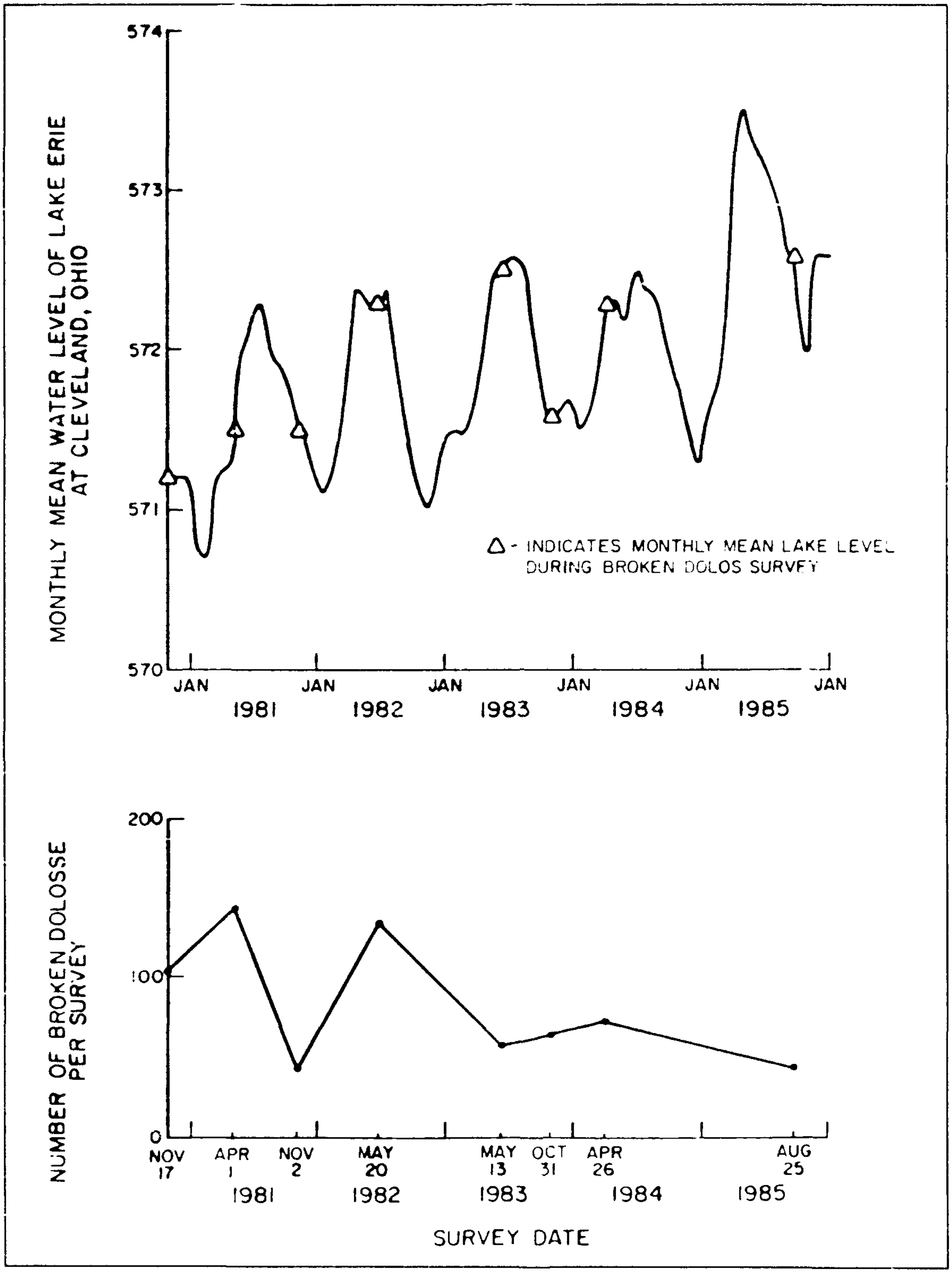


Figure 66. Comparison of lake level and number of broken dolosse per survey, Cleveland Harbor, Ohio

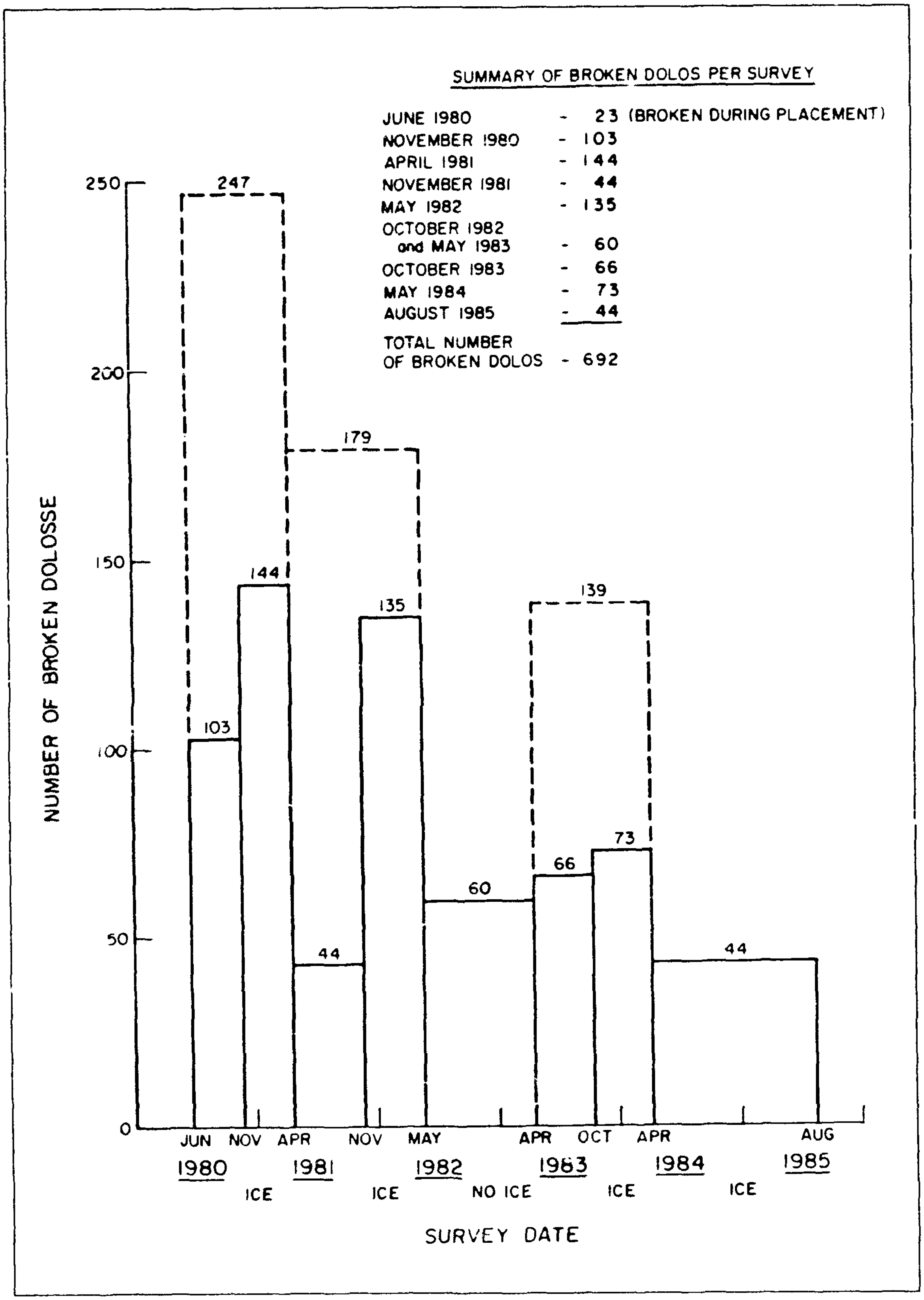


Figure 67. Summary of broken dolosse per survey at Cleveland Harbor

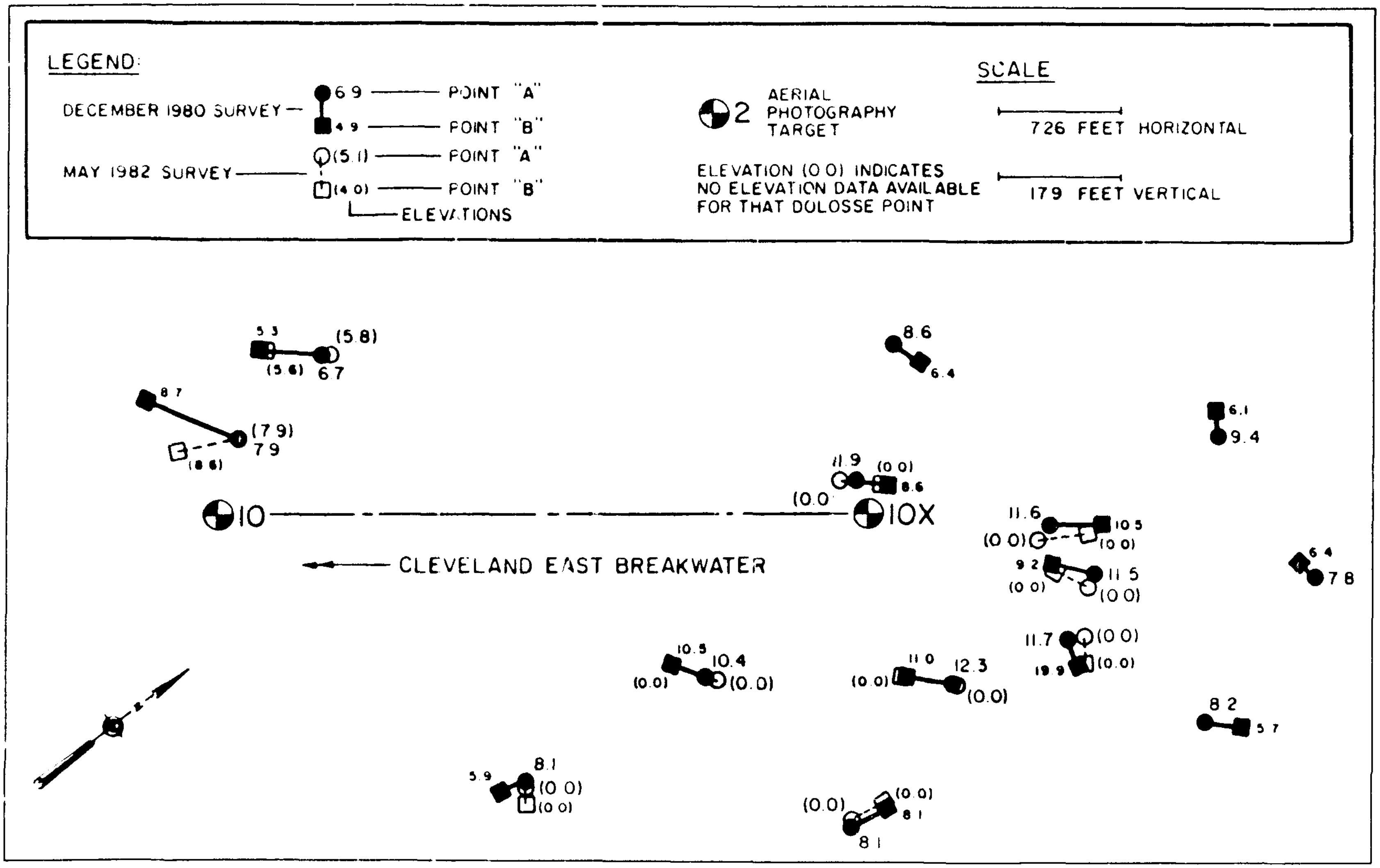


Figure 68. Results of dolos movement survey, Cleveland Harbor

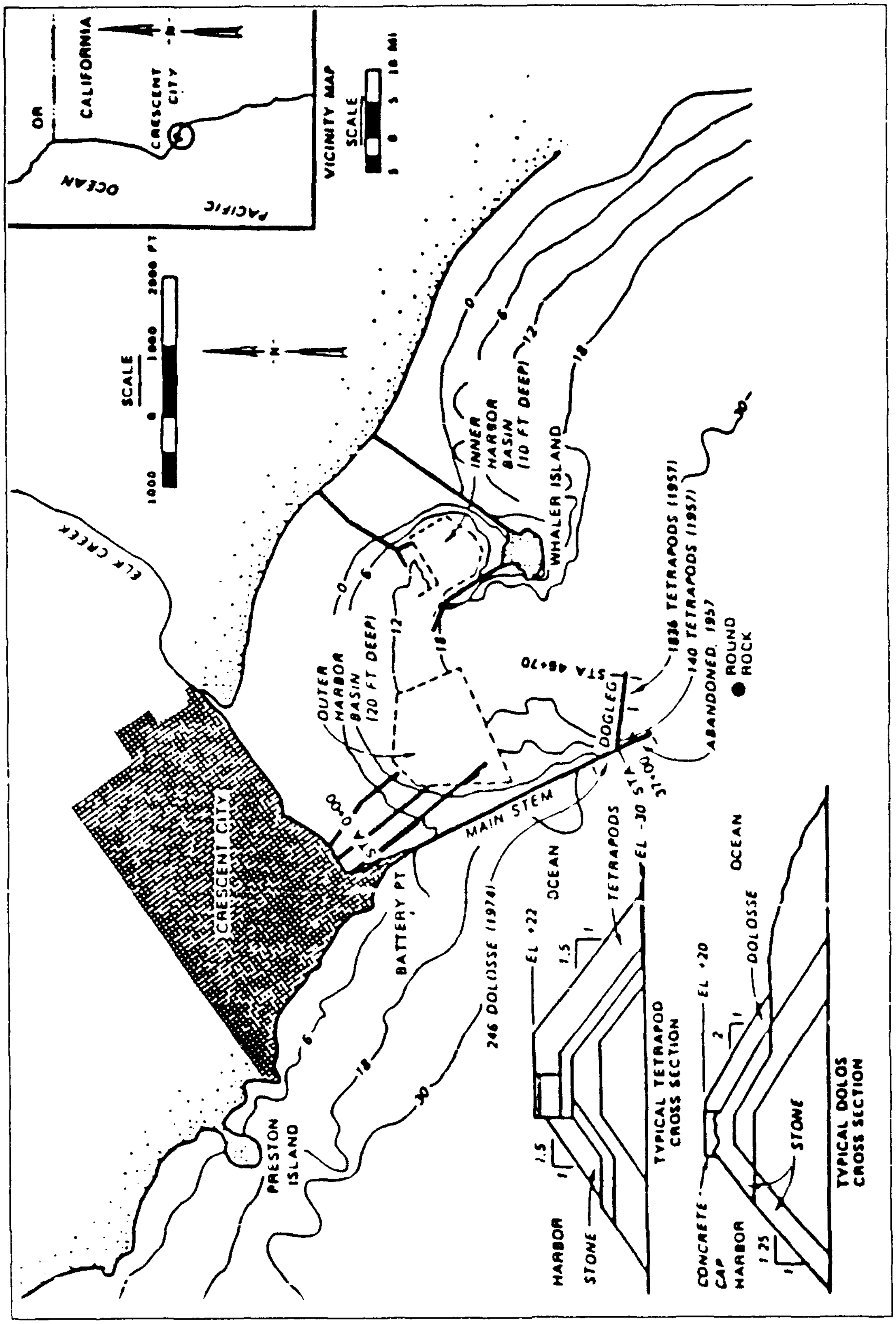


Figure 69. Cresent City Harbor Breakwater, California

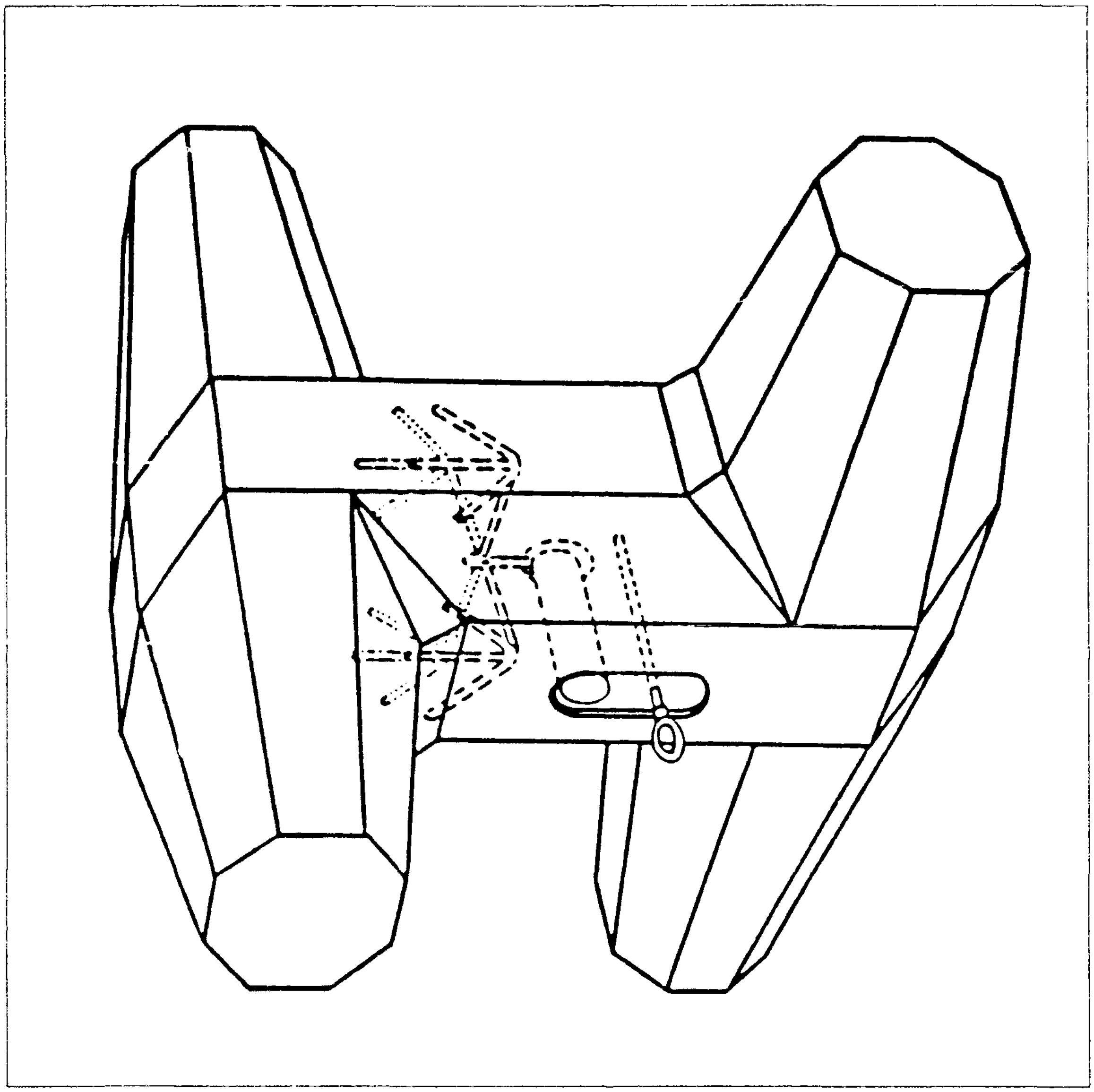


Figure 70. Dolos instrumentation at Cresent City Harbor